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Optimizing hybrid decentralized systems for sustainable urban drainage infrastructures planning

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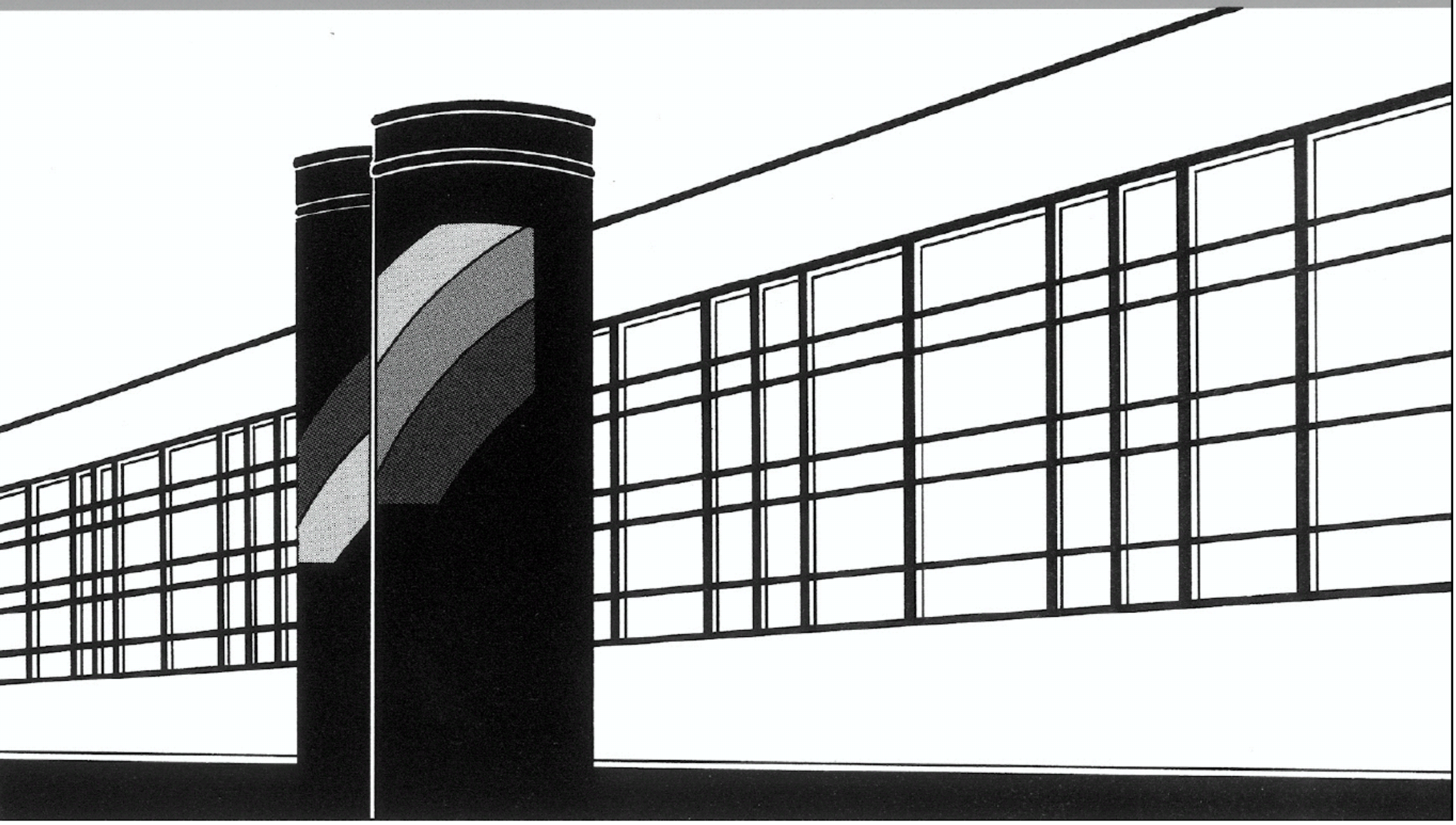

Heft 277 Amin Ebrahim Bakhshipour

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vorgelegt von

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Dedication

To my beloved hometown, Ahvaz

بعد از جنگ
هر کجای جهان
اگر دیدی دریا هست
اما آب نیست،
همانجا
زادرودِ تشنه من است
خسته به حضرتِ خوزا.

سیدعلی صالحی

After the war
wherever in the world
if you notice a sea
but no water,
that's where my thirsty river lays,
drained as the honored Khouzestan.

Seyed Ali Salehi

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Contents

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List of acronyms

CGI	Conventional Gray Infrastructure
DC	Degree of Centralization
DP	Dynamic Programming
EA	Evolutionary Algorithm
GA	Genetic Algorithm
GBI	Green-Blue Infrastructure
HGBGI	Hybrid Green-Blue-Gray Infrastructures
IDF	Intensity-Duration-Frequency
ILP	Integer Linear Programming
LCC	Life Cycle Costs
LP	Linear Programming
MCDM	Multi-Criteria Decision-Making
MOEA	Multi-Objective Evolutionary Algorithm
MOO	Multi-Objective Optimization
NLP	Non-Linear Programming
UDS	Urban Drainage Systems
SWMM	Storm Water Management Model
TOPSIS	Technique for Order of Preference by Similarity to Ideal Solution
TS	Tabu Search

Abstract

Motivation and Goal

Urban drainage systems (UDSs) are vital infrastructure systems that directly influence the public economy, health and welfare. Nowadays, it is becoming a well-accepted fact that UDSs must be not only reliable during normal loading conditions (design loads). They must also be resilient to extreme loading conditions (due to upcoming challenges such as climate change and urban growth). Furthermore, they must be sustainable in the long term to accomplish economic, environmental and social aims. Recent research criticizes the performance of conventional centralized underground systems (Gray Infrastructures) in coping with the aforementioned challenges. Therefore, a paradigm shift for design, rehabilitation and retrofit strategies of UDSs is inevitable. Beside a paradigm shift in design strategies, many authors and studies so far have suggested a transition from gray-only centralized UDSs to hybrid decentralized UDSs as the most promising urban water management approach.

A hybrid decentralized UDS can be obtained by taking two different strategies: (1) structural decentralization, and (2) hybridization. Structural decentralization strategies try to reduce the degree of centralization of a system by reducing the number of elements that are linked to it and interconnected. For example, by releasing water to different locations of water bodies. Hybridization in water engineering can be obtained by introducing two or more types of fundamentally different elements that acting together to accomplish a set of pre-defined design goals (e.g., flood prevention and rainwater treatment). In UDSs hybridization may be obtained through distributed measures such as green-blue infrastructures (GBIs).

GBIs are flexible and adaptable measures that provide several co-benefits besides flood risk reduction. Such co-benefits include water quality improvement (for infiltration systems), maintaining the natural water balance and urban climate improvement. Gray measures, on the other hand, are widely tested systems that show more resilience to cope with intense rainfall. Therefore, hybrid green-blue-gray infrastructures (HGBGIs) theoretically, can simultaneously combine reliability, resilience and acceptability of conventional gray infrastructures with the multi-functionality, sustainability and adaptability of GBIs.

Notwithstanding, considering all combinations of possible alternatives, conventional gray infrastructures and GBIs, and many (often) conflicting objectives (e.g., life cycle costs and sustainability) as well, to design new UDSs constructs a notably complex optimization problem. Obtaining the optimum layout of the pipe network considering the different degrees of (de-)centralization, sizing the sewers, selecting the type, size and location of GBIs are the sub-problems that need to be decided simultaneously for this aim. Each of these optimization sub-problems contains many decisions, as well as technical and hydraulic constraints.

As a response to the above-mentioned challenges, the main objective of this dissertation is to assist sustainable UDSs planning. The general aim is to develop algorithms, methods and tools that aid in the mathematical interpretation of the concepts of decentralization and hybridization. The contributions made culminate in a combined multi-objective optimization and multi-criteria decision-making platform to design hybrid decentralized UDSs.

To approach the main goal, it is structured into three sub-goals or objectives. Each sub-goal leads to the development of a contribution, as presented below. These contributions build upon each other. The main approaches include novel algorithms development, mathematical optimization formulation and multi-criteria decision-making platform development.

Contributions and Conclusions

1. Development of a framework for *decentralized UDSs optimization*

This contribution develops an algorithm based on graph theory, called the *hanging gardens algorithm* to generate all possible sewer layouts and to explore different DC. To form a simulation-optimization framework, an optimization engine is coupled with the proposed layout generator algorithm and with hydraulic simulation software (SWMM). This forms a non-linear complex optimization problem with one objective function, life cycle costs and many decisions, including the number and location of the outlets, layout configuration and the size of sewers.

To demonstrate the performance of the proposed framework it is then applied against a real case study, a section of the city of Ahvaz. Ahvaz is a totally flat city located in the southwest of Iran.

The results suggest that structural decentralization can significantly reduce the construction costs, pipe sizes and invert depths in comparison with the centralized layout; however, after a particular DC (optimal DC), more decentralization might lead to a diseconomy of scale. The optimal DC depends on the case study specifications and problem setup. Besides, results demonstrate that structural decentralization could increase the functional resilience in the system.

The key conclusion from this part is that the proposed model provides an appropriate tool to explore different DCs, generating realistic layouts and finding near-optimum solutions.

2. Development of a framework for the *optimization of hybrid decentralized UDSs*

The second contribution extends the simulation-optimization framework by considering HGBGI alternatives and different DC. The performance of the proposed framework is evaluated using the same case study.

The results show that GBIs could significantly diminish the life cycle costs of more centralized layouts. However, for the more decentralized layouts, the hybrid solutions are marginally more expensive than traditional solutions. The results also confirm the poor functional resilience of HGBGIs in comparison with conventional gray infrastructures in facing severe rainstorms. On the other hand, HGBGIs show better performance in environmental sustainability by a higher reduction in peak flow and higher storage and infiltrating capacity.

The key conclusion from this part is that the optimal DC depends on the objectives. It differs for construction costs, resilience and sustainability. Therefore, the optimization of new green-blue-gray UDSs should be done in a joint multi-objective optimization framework for better decision making.

3. Development of a platform for the *sustainable planning of hybrid decentralized UDSs*

The third contribution introduces a combined Multi-Objective Optimization and Multi-Criteria Decision-Making platform. Its goals are to (1) facilitate optimization of hybrid (de)centralized urban drainage infrastructures with many decisions and conflicting objectives, (2) to investigate the trade-offs between performance indicators (reliability, resilience and sustainability) and system configuration (network layout and DC), and (3) to lessen the conflicts between optimization analysts and decision-makers by involving them in different stages of the planning procedure.

For the sake of demonstration, the proposed framework is applied again to the case study of Ahvaz. The results demonstrate the significant role of the layout configuration and degree of centralization on the optimum hybrid green-blue-gray infrastructures. The layout configuration also can determine the structural resilience of the system. The results prove the capacity of the proposed platform in handling many decisions, objectives and indicators for solving an extremely complex optimization problem in a plausible time and delivering acceptable optimal scenarios.

The key conclusion over this contribution is that it is possible to manage many practical or technical concerns that usually cannot be regarded in general optimization frameworks but are crucial for decision-makers within the proposed framework. This helps to (1) decrease the conflicts, (2) enrich the results of optimization with valuable experience of practitioners, and (3) increase the buy-in to the optimization results.

In brief, the outcome of this dissertation contributes in bridging gaps in (1) defining the optimal degree of centralization (DC) for the various real-world urban drainage planning problems, (2) designing modern hybrid (de)centralized urban water infrastructures, and (3) solving an extremely hard optimization problem with many (often) conflicting objectives, decisions, technical and practical constraints.

Kurzfassung

Motivation und Zielsetzung

Städtische Entwässerungssysteme (Urban Drainage Systems: UDS) sind lebenswichtige Infrastruktursysteme die direkten Einfluss auf die Wirtschaft sowie die öffentliche Gesundheit und Wohlfahrt haben. Heutzutage ist allgemein anerkannt, dass UDS nicht nur unter normalen Belastungsbedingungen (Bemessungsfall) zuverlässig funktionieren müssen. Sie müssen auch unter extremen Belastungsbedingungen (aufgrund bevorstehender Herausforderungen wie Klimawandel, unkontrollierter Urbanisierung) widerstandsfähig sein. Außerdem müssen sie langfristig nachhaltig sein, um wirtschaftliche, ökologische und soziale Ziele zu erreichen. Jüngste Forschungsarbeiten bewerten die Leistung konventioneller zentralisierter, unterirdischer Systeme (Graue Infrastruktur) bei der Bewältigung der oben genannten Herausforderungen kritisch. Daher ist ein Paradigmenwechsel bei der Planung, Sanierung und Nachrüstung von UDS unvermeidlich. Neben einem Paradigmenwechsel bei den Entwurfsstrategien haben viele Autoren und Studien bisher einen Übergang von ausschließlich grauen, zentralisierten UDS zu hybriden, dezentralisierten UDS als den vielversprechendsten Ansatz für das städtische Wassermanagement vorgeschlagen.

Hybride dezentralisierte UDS können durch zwei verschiedene Strategien erreicht werden: (1) strukturelle Dezentralisierung und (2) Hybridisierung. Strukturelle Dezentralisierungsstrategien versuchen, den Zentralisierungsgrad eines Systems zu verringern, indem sie die Anzahl der Elemente reduzieren, die mit ihm und miteinander verbunden sind. Zum Beispiel durch mehrere Einleitstellen in ein Gewässer. Hybridisierung bezeichnet die Kombination von zwei oder mehr Arten von grundsätzlich unterschiedlichen Elementen, die zusammenwirken, um vordefinierten Ziele zu erreichen (z.B. Überflutungsvorsorge und Regenwasserbehandlung). In UDSs kann die Hybridisierung durch dezentrale naturnahe Maßnahmen der Regenwasserbewirtschaftung (Grün-blaue Infrastruktur GBI) erreicht werden.

GBIs sind flexible und anpassungsfähige Maßnahmen, die neben der Verminderung des Hochwasserrisikos mehrere Zusatznutzen bieten. Zu diesen Zusatznutzen gehören z.B. die Verbesserung der Wasserqualität (bei Versickerungsanlagen), die Annäherung an den natürlichen Wasserhaushalt und die Verbesserung des Stadtklimas. Graue Maßnahmen hingegen sind weithin erprobte Systeme, die eine größere Resilienz gegenüber intensiven Regenfällen aufweisen. Daher können hybride grün-blau-graue Infrastrukturen (Hybrid Green Blue Gray Infrastructure, HGBGI) theoretisch gleichzeitig die Zuverlässigkeit, Resilienz und Akzeptanz herkömmlicher grauer Infrastrukturen mit der Multifunktionalität, Nachhaltigkeit und Anpassungsfähigkeit von GBIs kombinieren.

Ungeachtet dessen stellt der Entwurf neuer UDS unter Berücksichtigung aller Kombinationen möglicher Alternativen, konventioneller grauer Infrastrukturen und GBIs und vieler (oft) auch gegensätzlicher Zielsetzungen (z.B. Lebenszykluskosten und Nachhaltigkeit) ein besonders komplexes Optimierungsproblem dar. Die optimale Auslegung des Rohrnetzes unter Berücksichtigung der verschiedenen Grade der (De-)Zentralisierung (Degree of Centralisation: DC), die Dimensionierung der Kanäle, die Auswahl des Typs, der Größe und des Standorts von GBI-Anlagen sind Teilprobleme, die gleichzeitig entschieden werden müssen. Jedes dieser Optimierungs-Unterprobleme enthält viele Entscheidungen sowie technische und hydraulische Einschränkungen.

Vor dem Hintergrund der oben genannten Herausforderungen besteht das Hauptziel dieser Dissertation darin, die nachhaltige Planung von UDS zu unterstützen. Das allgemeine Ziel ist die Entwicklung von Algorithmen, Methoden und Werkzeugen, die bei der mathematischen Interpretation der Konzepte der Dezentralisierung und Hybridisierung helfen. Die geleisteten Beiträge werden zusammengeführt zu einer kombinierten Plattform zur mehrdimentionalen Optimierung und multi-kriteriellen Entscheidungsfindung bei der Planung hybrider dezentraler UDS.

Um dem Hauptziel näher zu kommen, wurde es in drei Unterziele unterteilt. Jedes Unterziel führt zur Entwicklung der unten dargestellten Beiträge. Diese Beiträge bauen aufeinander auf. Sie umfassen die Entwicklung neuartiger Algorithmen, die mathematische Optimierungsformulierung und die Entwicklung einer multikriteriellen Entscheidungsplattform.

Beiträge dieser Arbeit und Schlussfolgerungen

1. Entwicklung eines Frameworks zur *dezentralen UDS-Optimierung*

In diesem Beitrag wird ein auf der Graphentheorie basierender Algorithmus entwickelt, der als "Hanging Gardens Algorithm“ bezeichnet wird, um alle möglichen Kanallayouts zu generieren und verschiedene DC zu untersuchen. Um einen Simulations-Optimierungs-Framework zu bilden, wird ein Optimierungsalgorithmus mit dem Layout-Generator-Algorithmus und mit der Simulationssoftware (SWMM) gekoppelt. Dies bildet ein nichtlineares komplexes Optimierungsproblem mit einer Zielfunktion, die Lebenszykluskosten, und viele Entscheidungen, einschließlich der Anzahl und Lage der Auslässe, der Layout-Konfiguration und der Größe der Kanäle.

Um die Leistungsfähigkeit des vorgeschlagenen Frameworks zu demonstrieren, wird er an einer realen Fallstudie, einem Teilgebiet der Stadt Ahvaz, im Südwesten des Iran, angewendet. Die Topografie des Stadtgebietes ist völlig flach.

Die Ergebnisse deuten darauf hin, dass Dezentralisierung der Systemstruktur die Baukosten, Kanaldurchmesser und Sohltiefen im Vergleich zum zentralisierten Layout erheblich reduzieren kann; oberhalb des Optimums könnte jedoch eine weitere Dezentralisierung zu ungünstigeren Lösungen führen. Der optimale DC hängt sehr stark von den fallspezifischen Bedingungen und der Problemstellung ab. Außerdem zeigen die Ergebnisse, dass eine Dezentralisierung der Struktur die funktionelle Resilienz des Systems erhöhen könnte.

Die wichtigste Schlussfolgerung aus diesem Teil ist, dass das vorgeschlagene Modell ein geeignetes Instrument zur Untersuchung verschiedener DC darstellt, um realistische Layouts zu generieren und nahezu optimale Lösungen zu finden.

2. Entwicklung eines Rahmens für *die Optimierung hybrider dezentralisierter UDS*

Der zweite Beitrag erweitert deas Simulations-Optimierungs-Framework um die Berücksichtigung von HGBGI-Alternativen und verschiedenen DC. Die Leistung des vorgeschlagenen Ansatzes wird anhand derselben Fallstudie bewertet.

Die Ergebnisse zeigen, dass GBIs die Lebenszykluskosten von stärker zentralisierten Layouts erheblich senken könnten. Für die stärker dezentralisierten Layouts sind die hybriden Lösungen jedoch geringfügig teurer als konventionelle Lösungen. Die Ergebnisse bestätigen auch die geringe funktionale Resilienz von HGBGIs im Vergleich zu herkömmlichen grauen Infrastrukturen bei Extremniederschlägen. Auf der anderen Seite zeigen HGBGIs eine bessere Leistung in Bezug auf die Umweltauswirkungen durch eine weitergehende Reduzierung des Spitzenflusses und eine höhere Speicher- und Infiltrationskapazität.

Die wichtigste Schlussfolgerung aus diesem Teil ist, dass die Optimierungsziele Kosten, Resilienz und Nachhaltigkeit zu unterschiedlichen Ergebnissen für den optimalen Grad der Dezentralisierung führen.. Daher sollte die Optimierung neuer grün-blau-grauer UDS in einem gemeinsamen, mehrere Ziele verfolgenden Optimierungsrahmen für eine bessere Entscheidungsfindung erfolgen.

3. **Entwicklung einer Plattform für *die nachhaltige Planung von hybriden dezentralisierten UDS***

Der dritte Beitrag stellt eine kombinierte Plattform für eine mehrdimensionale Optimierung und multi-kriterielle Entscheidungsfindung vor. Ihre Ziele bestehen darin, (1) die Optimierung hybrider (de)zentralisierter städtischer Entwässerungsinfrastrukturen mit vielen Entscheidungen und Zielkonflikten zu erleichtern, (2) die Kompromisse zwischen Leistungsindikatoren (Zuverlässigkeit, Belastbarkeit und Nachhaltigkeit) und Systemkonfiguration (Netzlayout und DC) zu untersuchen und (3) die Konflikte zwischen Optimierungsanalytikern und Entscheidungsträgern zu verringern, indem sie in verschiedene Phasen des Planungsverfahrens einbezogen werden.

Zur Demonstration wird der vorgeschlagene Rahmen ebenfalls auf die Fallstudie von Ahvaz angewandt. Allerdings steigen die Lebenszykluskosten exponentiell mit einer Zunahme der Gesamtnachhaltigkeit. Die Ergebnisse zeigen auch die bedeutende Rolle der Layout-Konfiguration und des Grades der Zentralisierung bei den optimalen hybriden grün-blau-grauen Infrastrukturen. Die Layout-Konfiguration kann auch die strukturelle Belastbarkeit des Systems bestimmen. Die Ergebnisse beweisen die Fähigkeit der vorgeschlagenen Plattform, viele Entscheidungen, Ziele und Indikatoren zu handhaben, um ein extrem komplexes Optimierungsproblem in einer plausiblen Zeit zu lösen und akzeptable optimale Szenarien zu liefern.

Die wichtigste Schlussfolgerung aus diesem Beitrag ist, dass es mit dem vorgeschlagenen Ansatz möglich ist, viele praktische und technische Aspekte zu berücksichtigen, die in früheren Optimierungsansätzen nicht gemeinsam betrachtet werden konnten, die aber für Entscheidungsträger innerhalb des vorgeschlagenen Rahmens von entscheidender Bedeutung sind. Dies trägt dazu bei, (1) die Konflikte zu verringern, (2) die Ergebnisse der Optimierung mit wertvollen Erfahrungen von Praktikern anzureichern und (3) die Zustimmung zu den Optimierungsergebnissen zu erhöhen.

das Ergebnis dieser Dissertation trägt dazu bei, Lücken zu schliessen (1) bei der Definition des optimalen Zentralisierungsgrades für verschiedene reale Probleme der städtischen Entwässerungsplanung, (2) beim Entwurf moderner hybrider (de)zentralisierter städtischer Wasserinfrastrukturen und (3) bei der Lösung eines extrem harten Optimierungsproblems mit vielen (oft) widersprüchlichen Zielen, Entscheidungen, technischen und praktischen Einschränkungen.

Chapter 1. Introduction

1.1 Motivation and relevance for decentralized hybrid drainage systems

Urban drainage systems (UDSs) are vital urban infrastructures that directly influence the public economy, health and welfare. In most parts of the world, today's wastewater and stormwater management systems rely heavily on network-based infrastructures, involving long-distance channels or pipelines to transport wastewater or stormwater between urban areas and treatment facilities or water bodies [1–3]. Their main task is to drain wastewater and stormwater properly from urban areas. If they fail it will cause pollution, health risks, inconvenience, damage and flooding [4].

Recent research castigates the performance of traditional UDSs that are based on only gray infrastructures (e.g. pipe networks, storage tanks and centralized WWTPs) in coping with upcoming challenges such as climate change, urban growth, and providing long-term sustainability [1, 5–8]. The main reason is the lack of capital, especially in developing countries, where most governments do not have enough resources to build, maintain and rehabilitate such infrastructures [9]. In addition to economic reasons, other concerns question the sustainability of centralized infrastructures:

- **Environmental-ecological concerns** such as hydrological disruption, groundwater depletion, downstream flooding, receiving water quality degradation, channel erosion and stream ecosystem damage [10, 11]
- **The substantial risk** posed by the failure of centralized systems [8]
- **limited adaptivity** to rapid change and high uncertainty in a developing country context [12, 13]
- **System vulnerability** to climate change [12, 14]
- **Inappropriate water resource utilization and failing to address human livability adequately** during the infrastructure life cycle [2, 9, 10]

To deal with the abovementioned challenges, recent investigations suggest a transition from centralized urban water management to hybrid decentralized schemes [7, 8, 15, 16]. A hybrid decentralized UDS can be obtained by taking two different strategies: (1) structural decentralization, and (2) hybridization.

Decentralized solutions

Structural decentralization strategies try to reduce the degree of centralization of a system by reducing the number of elements that are linked to it and interconnected [7]. For example, by releasing water to different locations of water bodies.

The transition of traditional urban water systems towards decentralized solutions has significant effects on the remaining central water networks that need a comprehensive assessment [17]. However, in developing countries, where centralized infrastructures do not exist, there is also a chance to 'leapfrog' that centralized step directly to hybrid decentralized solutions. Leapfrogging theory proposes that developing countries may be able to leapfrog older versions of technology and avoid developed countries' path to industrialization with its environmentally disgraceful legacy [12, 18].

To find the optimal degrees of centralization (DC) and layout configuration, there is a need for tools and methodologies to generate all possible UDS layouts with arbitrary DC to be coupled with optimization engines. However, a brief overview of the literature (presented in Chapter 3) will show that only a few approaches are available for generating and optimizing decentralized urban drainage alternatives, which are still far from real applications.

Hybrid Solutions

A hybrid system in water engineering can be defined as a system with two or more types of elements (infrastructures) acting together to accomplish a set of pre-defined design goals. In the case of sewage collection systems, hybridization could be obtained through a modular approach. Instead of investing large sums for treatment plants and connecting a heavily centralized network of pipes to it, several smaller plants or on-site measures (e.g., on-site greywater reclamation system with small-scale MBRs) can be employed [2, 13].

For separate stormwater management systems, besides decentralized methods that release stormwater to different locations of water bodies, hybridization may be obtained through decentralized (distributed) measures such as green-blue infrastructures (GBIs). Examples of GBIs are detention/retention ponds, constructed wetlands, rain gardens, green roofs, permeable pavements, infiltration basins or rain barrels [19, 20].

GBIs are flexible and adaptable measuress that provide several co-benefits besides flood risk reduction. These co-benefits include water quality improvement, water quantity reduction, flood mitigation, recharging groundwater, water harvesting, restoring the hydrologic characteristics of the site, increasing urban amenity and alleviating the urban heat island effect [11, 19, 21–24].

The advantages of including GBIs for retrofitting purposes have been widely discussed and acknowledged, mainly in developed countries. However, they cannot fully replace conventional gray infrastructures, especially in developing countries and for planning new infrastructures [25]. The reasons are lack of space in highly urbanized areas, socio-economic factors, the lack of environmental awareness, public acceptance, and GBI's inability to control extreme events [5, 11].

In brief, gray measures are widely tested systems that show more resilience to cope with intense rainfall, while GBIs offer multiple benefits such as adaptability and sustainability [26, 27]. Many authors and studies so far have suggested hybrid green-blue-gray infrastructures (HGBGIs) as the most promising urban water management approach that can simultaneously combine reliability, resilience and acceptability of conventional pipe networks with multi-functionality, sustainability and adaptability of GBIs [26, 28, 29].

For the optimal selection of type, location and size of GBIs, numerous optimization and decision-making tools and methods have been developed so far in the literature. The focus of methods to optimize GBIs is mainly finding optimum retrofitting strategies through combing GBIs with existing gray infrastructures [26]. Notwithstanding, the review of previous studies in Chapter 4 will reveal that there is no tool or methodology for identifying the effects of the interaction between gray and green-blue infrastructures in the design phase of UDSs.

The need for multi-objective decision support

UDSs are traditionally designed using hydraulic reliability-based approaches that try to assure a sufficient hydraulic capacity for the conveyance of some design flow [4, 30]. Nowadays, it is becoming a well-accepted fact that other objectives such as socio-ecological sustainability, resilience and adaptability need to be considered in the planning or rehabilitation phase of urban water infrastructures [26, 29, 31, 32].

The literature has widely addressed the hydraulic reliability-based approaches that mainly combine mathematical simulation models with optimization/decision-making methods to design, rehabilitate, or retrofit UDSs [30]. However, even the performance of conventional reliability-based design approaches has frequently been disputed as well, and a paradigm shift for design, rehabilitation and retrofit strategies of UDSs is suggested by researchers to cope with the aforementioned challenges, and to guarantee the ongoing reliability, resilience and sustainability of service provision [33, 34].

That means UDSs should be designed according to service objectives:

- *Reliable* during normal loading conditions to minimize failure frequency
- *Resilient* to extreme loading conditions to lessen the span and extent of the failure
- *Sustainable* in the long term to accomplish economic, environmental and social aims [35–37]

Identified Challenges

To conclude, future UDSs need to be reliable, resilient and sustainable. HGBGIs seem to be the most supportive approach to achieve these goals. Notwithstanding, considering all combinations of possible alternatives, gray and green-blue infrastructures and many objectives as well, to design new UDSs constructs a notably complex optimization problem. Obtaining the optimum layout of the pipe network considering the different DC, sizing the sewers, selecting the type, size and location of GBIs are the sub-problems that need to be decided simultaneously for this aim. Each of these optimization sub-problems contains many decisions, technical and hydraulic constraints. Besides these, there are different objectives that, in some cases, conflict with each other [38], and increase the dimension of problem complexity.

The review of previous studies in Chapter 5 will reveal that the available literature has solved this problem either by isolating the objectives or the alternatives. Besides, the literature mostly address rehabilitating/retrofitting strategies where the number of decisions and problem complexity is much lower than designing new systems. Moreover, attributes and relationships between the following performance indicators: reliability, resilience and sustainability, UDS configuration and life cycle costs (LCC) need to be explored [36, 39]. Finally, a framework is needed to facilitate the procedure of decision-making from all non-dominated solutions obtained by multi-objective optimization and involve stakeholders in decision-making to decrease the conflicts between them and academic researchers.

This thesis aims to address the challenges mentioned above.

1.2 Goal, approach and objectives

The **main goal** of this doctoral thesis is to assist sustainable UDSs planning. The general aim is to develop algorithms, methods and tools that aid in the mathematical interpretation of the concept of decentralization to establish a generic multi-objective decision-making platform for the optimal design of hybrid decentralized UDSs, specifically stormwater management systems.

To **approach** the main goal, it has been discretized into three sub-goals or objectives. Each sub-goal leads to the development of an algorithm, method, or framework that is used as the central core to add in the next sub-goal, as shown in Figure 1.1 (see the connections between Chapters 3 to 5). The main approaches cover

- novel algorithm development for decentralized UDS layout and hybrid UDS schemes generation,
- mathematical single and multi-objective optimization formulation using different optimization engines,
- multi-criteria decision support platform development using multi-criteria decision support techniques.

As mentioned above, this study has three **main objectives**, as presented below.

Objective 1: Develop a framework for optimizing structurally **decentralized** UDSs.

- *Objective 1.1*: Develop a tailor-made algorithm to generate all possible sewer layouts with an arbitrary DC that systematically satisfies all constraints of designing an urban drainage layout. The mathematical representation of generated networks must be close to real sewer systems.
- *Objective 1.2*: Formulate the optimization problem and develop the cost functions.
- *Objective 1.3*: Demonstrate the performance of the proposed algorithm in enumerating all different DC and generating realistic layouts by using it to optimize the stormwater collection network of a real case study, a section of the city of Ahvaz.

Objective 2: Develop a framework for the optimization of **hybrid** green-blue-gray UDSs.

- *Objective 2.1*: Develop an algorithm to generate all possible combinations of the gray infrastructures and green-blue infrastructures (distributed measures).
- *Objective 2.2*: Formulate the optimization problem and develop the cost functions for the distributed measures.
- *Objective 2.3*: Evaluate the utility of the developed framework by applying to the case study.

Objective 3: Develop a generic optimization framework for designing sustainable UDS that can handle **many objectives** and **many decision** variables in **plausible computation** time.

- *Objective 3.1*: Propose reliability, resilience and sustainability indicators to assess the performance of different UDS schemes.
- *Objective 3.2*: Formulate the multi-objective optimization problem, considering reliability, resilience and sustainability as objectives and gray and green-blue infrastructures as decision variables.
- *Objective 3.3*: Identify and assess the main performance trade-offs between DC, layout configuration and performance indicators by applying the proposed framework to the case study.

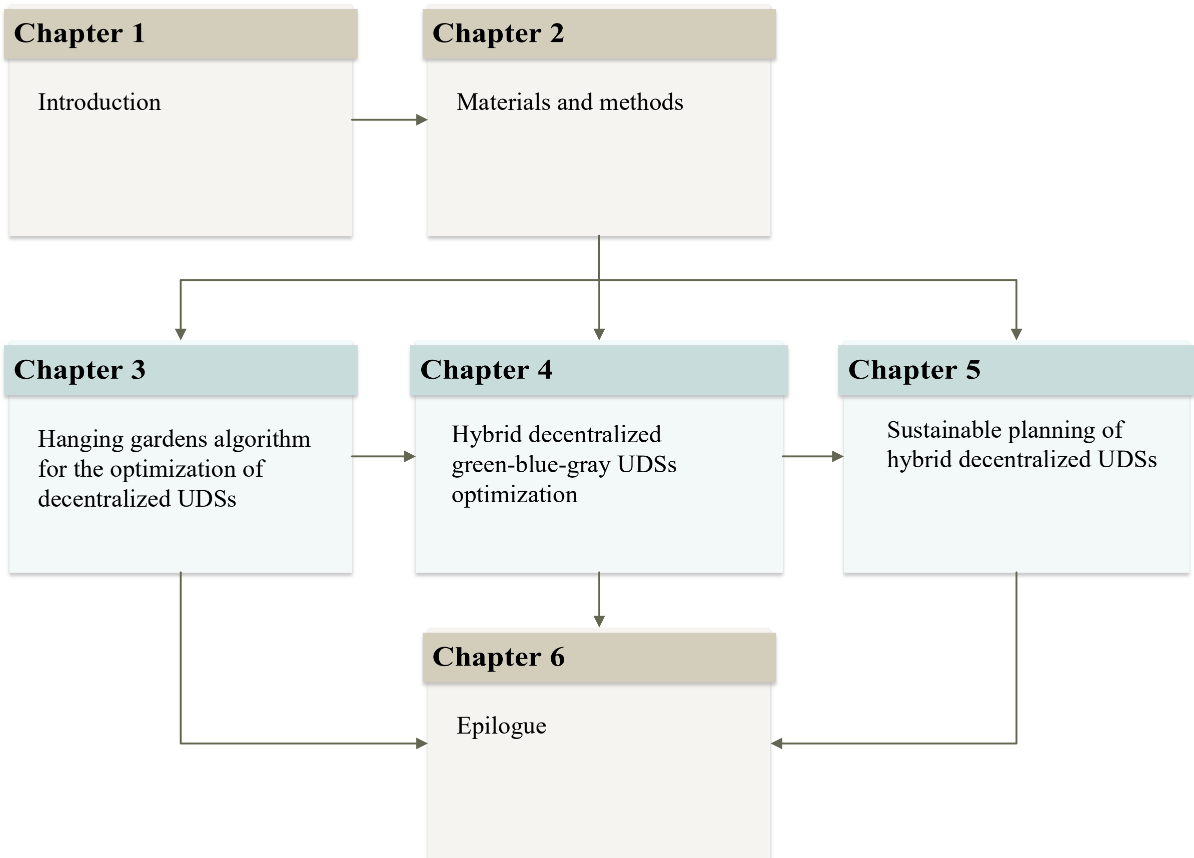


Figure 1.1: *Thesis structure and interactions between thesis chapters*

Completing these objectives will answer the research questions shown in Table 1.1.

Table 1.1: *The relationship between research objectives, research questions and the chapters contain the thesis contributions*

Objective	Research Question	Chapter
1.1	Is it possible to generate all possible realistic UDS layouts from totally centralized to totally decentralized only using the primary data such as street alignments and the possible location of outlets?	**3**
1.2, 2.1 & 2.2	How can mathematical optimization frameworks help to determine the optimal number, location of outlets simultaneously with the layout configuration, the size of pipes, the type, size and location of distributed measures (e.g., GBIs)?	**3 & 4**
1.3	How does DC influence the construction costs and performance of UDSs?	**3**
2.3	Can HGBGIs compete with conventional gray infrastructures economically?	**4**
2.3	How do the layout configuration of pipe networks and the DC affect the economic efficiency of HGBGIs?	**4**
2.3	How do HGBGIs compare with gray systems in terms of resilience and sustainability?	**4**
3.1	How can we assess different UDS schemes for reliability, resilience and sustainability?	**5**
3.2	How to consider many objectives and many decisions in the optimization framework to reach convergence in a plausible time?	**5**
3.3	What are the main trade-offs involved between performance indicators, construction costs, DC and layout configuration in the optimal hybrid decentralized UDSs?	**5**

1.3 Contributions and outline

This thesis is arranged into six chapters that act together, as shown in Figure 1.1. Chapters 1 and 2 do not contain any novelties or scientific contributions. The main contributions are being presented in Chapters 3 to 5. In this thesis, the state of the art is distributed in Chapters 3 to 5, as each chapter regards a different aspect of designing UDSs.

Chapter 1, the present chapter, describes the background and motivation of the research project, the main goal and objectives to be fulfilled, and the thesis structure.

Chapter 2 introduces the general guidelines for designing UDSs, their governing equations, technical and practical constraints, simulation software, green-blue infrastructures, fundamentals of optimization algorithms (single and multi-objectives) and multi-criteria decision support techniques.

In chapter 3, an algorithm based on graph theory called the *hanging gardens algorithm* is developed to generate all possible sewer layouts and to explore different DC. To demonstrate the performance of the proposed algorithm in enumerating all different DC and generating realistic layouts, the algorithm is coupled with an optimization engine to optimize the stormwater collection network of a section of the city of Ahvaz in Iran. The number and location of outlets, the layout configuration of each part and the size of pipes are used as optimization variables to minimize costs subject to hydraulic and feasibility constraints.

Chapter 4 presents a simulation-optimization framework to optimize UDSs considering HGBGI alternatives and different DC. The proposed framework begins with characterizing the site under design. Then, different layouts with different DC are generated and hydraulically designed using the hanging gardens algorithm introduced in Chapter 3. After introducing the feasible GBI to the model, a second optimization is performed to find the optimum distribution of GBIs in a way that minimizes the total LCC of GBIs and pipe networks. The performance of the proposed framework is evaluated using the same case study.

Chapter 5 introduces a generic framework for the multi-objective optimization of hybrid UDSs that considers the DC, layout configuration, a range of distributed measures (GBIs) and technical and construction constraints for investigating the solutions that represent near-global optimal trade-offs among often competing objectives. The proposed framework facilitates the stakeholders' participation in decision-making to decrease the conflicts between them and academic researchers by involving them in different stages. With this framework, the trade-offs between life cycle costs, DC, reliability, resilience and sustainability are also investigated. The utility of the approach is demonstrated again on Ahvaz case study.

Chapter 6 summarizes the overall goal and objectives, reviews the main findings and the key contributions of the work, discusses the conclusions conferred along with the thesis, and grants recommendations for future research and application.

Relation to Published Works

The contributing chapters of this thesis, Chapters 3 to 5, are based on several publications, including Bakhshipour et al. (2018) [40], Bakhshipour et al. (2019a) [41], Bakhshipour et al. (2019b) [42], Bakhshipour et al. (2021a) [43], and Bakhshipour et al. (2021b) [44]. The sources are declared at the very beginning of each chapter.

Chapters 1, 2 and 6 may contain similar formulations from the above-mentioned publications. Nevertheless, they do not contain any scientific innovations and contributions. Therefore, precise identification is disregarded in these chapters.

Chapter 2. Fundamentals

Summary

In this chapter, a brief introduction to all fundamentals that are relevant to understanding this thesis is given. Section 2.1 gives an introduction about UDSs, the conventional approach to design them and the shortcomings of this approach. The focus of section 2.2 is designing the layout of the UDS by presenting an introduction about graph theory. Section 2.3 is about the hydraulic design of different components of a UDS. This section introduces the flow modeling approaches in the different parts of UDSs, governing equations and the used simulation software (SWMM). Finally, section 2.4 gives a brief introduction to decision making with optimization by summarizing different techniques for optimization and decision support systems techniques.

2.1 Urban drainage systems

Urban drainage systems (UDSs) are one of the critical infrastructures in urban areas that play an essential role in the public's health and safety [45]. Wastewater and stormwater are two types of water that require drainage. If wastewater was not drained properly, it could endanger the environment and could cause health risks. If stormwater was not drained appropriately, it would cause additional health risks, public inconvenience and damage [4]. As all proposed frameworks and methods within this thesis are applied to design stormwater drainage systems, this section is aimed at describing the present state-of-the-art in storm drainage systems modeling and design.

The traditional primary objective of storm drainage systems has been to collect extra stormwater from surfaces rapidly, transport it typically via a network of pipes, and dispose it into the nearest receiving water body [4, 45]. This approach results in many adverse impacts on the environment. Hydrological disruption, groundwater depletion, downstream flooding, pollution in water bodies, and stream ecosystem damage are a sample of degrading legacies of gray infrastructures [11, 12].

Nowadays, it is becoming a well-accepted fact that other objectives such as socio-ecological sustainability, resilience, and adaptability need to be reflected in the planning or rehabilitation phase of urban water infrastructures [26, 29, 31, 32]. Therefore, various sustainable stormwater management measures have been recommended to mitigate the problems mentioned above in more environmentally-friendly ways [20, 46–48].

These multi-functional and decentralized (distributed) measures are generally referred to as low-impact development, best management practices, green infrastructures, green-blue infrastructures (GBI), or water sensitive urban design [20, 49, 50]. Notwithstanding, the distinct terminologies differ lightly in their meanings due to their histories [49]. In this thesis, I use the term GBI. Some common GBI practices are bio-retention cells, infiltration trenches, stormwater wetlands, wet ponds, permeable pavements, green roofs, filter strips, sand and gravel filters and rain barrels [11, 20, 51].

The main objectives concerning different receptors that must be followed in the planning and management of modern storm drainage systems are as follows [45]:

- **Public:** protection of public and vital infrastructures during storms
- **Environment:** conserving the operation of the natural drainage system and protecting the environment
- **Economic:** design and operation of these systems, in a way that minimizes damage costs, installation costs and operational/maintenance costs.

The main components of modern drainage systems are (1) gray infrastructures (e.g., the network of pipes, pumping facilities, storage tanks and treatment facilities), and (2) green-blue infrastructures that need to be designed simultaneously to achieve the design objectives as mentioned above.

The conventional approach to design UDSs and related shortcomings

The design of an urban drainage system needs to solve two successive sub-problems [52]:

1. **Designing the layout.** This involves determining the configuration of pipes in the network and the locations of facilities such as pump stations, GBIs and storage tanks.
2. **Sizing the network's components.** This involves sewer diameters and installation depths, size of GBIs and storage tanks, as well as the pumping facilities.

Designing the layout

Conventional approaches to design UDSs include some fundamental stages. Firstly, the contributing area is defined and marked on a topographical map. The possible outlet(s) is identified. Next, a preliminary horizontal alignment (layout) is produced. As pipe networks are supposed to collect wastewater and stormwater gravitationally, the designer could rely on the topography of the area and follow natural ground slopes in the direction of the determined outlet. This approach might lead to a near-optimal layout depending on the designer's experiences and the steepness of the area. If suitable natural slopes are exploited appropriately, the obtained layout can result in reducing the construction cost of the system by reducing pipe sizes, excavation volumes of the sewers and the need for pumping facilities.

In flat areas, the problem is thoroughly dissimilar and challenging to solve. There is no noteworthy alteration in topography elevations; subsequently, the designer cannot see and trace prominent natural ground slopes to a distinguished outlet. In such areas, there are often countless possibilities for the connectivity of the sewers and the location of the outlet(s). In contrast with steep areas, engineering experiences and judgments are not adequate to design the sewer layout of flat areas. In these areas, the number of possible layouts exponentially increases with the number of conduits. The lack of natural slopes in the network makes its design specifications and its construction and operational costs exceedingly sensitive to the configuration of the layout [52–54]. Therefore, it is beneficial to utilize optimization methods to generate the near-optimal layout as conventional approaches are not very efficient.

Sizing the network's components

After drawing the layout, the size of pipes, their slopes as well as size and location of distributed measures (e.g., GBIs, pumping and storage facilities) must be calculated based on estimated flows from the contributing area in a way that satisfies all technical and practical design constraints [55].

The sizing procedure is accomplished by first choosing a suitable design storm. The choice of design storm return period determines the degree of protection from stormwater flooding provided by the system. This protection should be related to the cost of any damage or disruption that might be caused by flooding. As these cost-benefit studies are rarely conducted in practice for regular urban drainage projects, a decision on the design storm return period is mostly made based on judgment and standards [4].

The so-called rational method is the most common approach to size stormwater or combined collection sewers. The pipes are sized such that they convey the rational design discharge flowing just full [56]. The shortcomings of this approach prevent providing optimal UDSs, as declared in the following paragraphs.

Firstly, wastewater and stormwater flow might vary dramatically with the time of day and during storm conditions. However, in the rational method, flow conditions are treated as steady. This method only produces worst-case design flow and not a hydrograph of flow against time [4]. For more accurate flow modeling in the pipes, it is necessary to solve full 1D Saint Venant equations. As is described in section 2.3, there is a complicated relationship between depth and flow-rate in unsteady flow in a part-full pipe. While a storm wave flows through a sewer system, it attenuates (it spreads out and the peak reduces). The relationship between flow-rate (or depth) and time cannot be accurately predicted without taking this effect into account. Besides, accurate simulation of unsteady flow might prevent overdesigning that would often result from the assumption that waves did not change that form [56].

Moreover, this approach is based on a trial and error procedure. The size of system components is suggested and then verified for compliance with technical and hydraulic constraints. In the case of a violation of the constraints, a new system configuration is proposed. This procedure is iterated until acceptable performance is obtained. This approach only provides one or a maximum of several acceptable scenarios that are not necessarily optimal. Besides not providing optimal solutions, the trial and error approaches are time-consuming and require much human effort. Therefore, recent academic literature has tried to utilize mathematical optimization methods to (1) deliver optimal solutions and (2) to make the design procedure systematic [57].

These approaches mainly enjoy simulation-optimization procedures in which (1) a mathematical model is employed to mimic the real sewer network, and (2) an optimization engine is linked to the mathematical model to aid the decision making by automatically generating and evaluating a large number of scenarios [57]. All the proposed framework within this thesis are based on this approach. The fundamentals that are needed to understand these frameworks including (1) mathematical representation of the urban drainage layouts and corresponding constraints, (2) mathematical models to simulate the flow in the UDSs, and (3) an introduction to mathematical optimization methods, are given in the following sections.

2.2 Mathematical representation of urban drainage layouts

The layout of a UDS can mathematically be presented as a graph with specific properties. To address this topic, it is required to review some basic concepts and terminologies in graph theory. Graph theory is a branch of discrete mathematics that studies the principles of mathematical expression of graphs. Focusing on the scope of this thesis, some principles of graphs, mostly taken from [58], are summarized as follows:

- **Undirected graph:** Two finite sets define an undirected graph G: (1) a non-void set X of elements called vertices, (2) a set E of elements called edges (Figure 2.1a). Here, the vertices represent manholes and the edges represent sewer lines. The number of X and E is respectively denoted by n and m.
- **Directed graph:** Two finite sets define a directed graph, abbreviated to a digraph: (1) a non-empty set X of vertices and (2) a set A of directed edges, with an ordered pair $(x;\ y)$ where y is called the head and x is called the tail (Figure 2.1b). In a classification, digraphs may be defined as cyclic digraphs (including loops or double edges) and acyclic digraphs (in the absence of loops). The latter is also referred to as the tree digraph (Figure 2.1c).
- **Connected and Disconnected Graph:** A graph G is said to be connected if any two vertices of this graph are linked by a path in G (Figure 2.1a–c). Otherwise, the graph is disconnected.
- **Tree*:*** An undirected graph G is said to be a tree if any two vertices are connected by exactly one path, or equivalently a connected acyclic undirected graph.
- **Spanning Trees:** A spanning tree of graph G is a tree sub-graph, including all vertices. By definition, a connected graph G has (at least) one spanning tree.
- **Root:** The root of a digraph G is a vertex r, like vertex 3 in Figure 2.1c, such that there is for any vertex x of G a directed path from x to r. In general, a digraph may have several roots.
- **Arborescence:** An arborescence is a digraph which has one root and of which the underlying graph is a tree (Figure 2.1c).

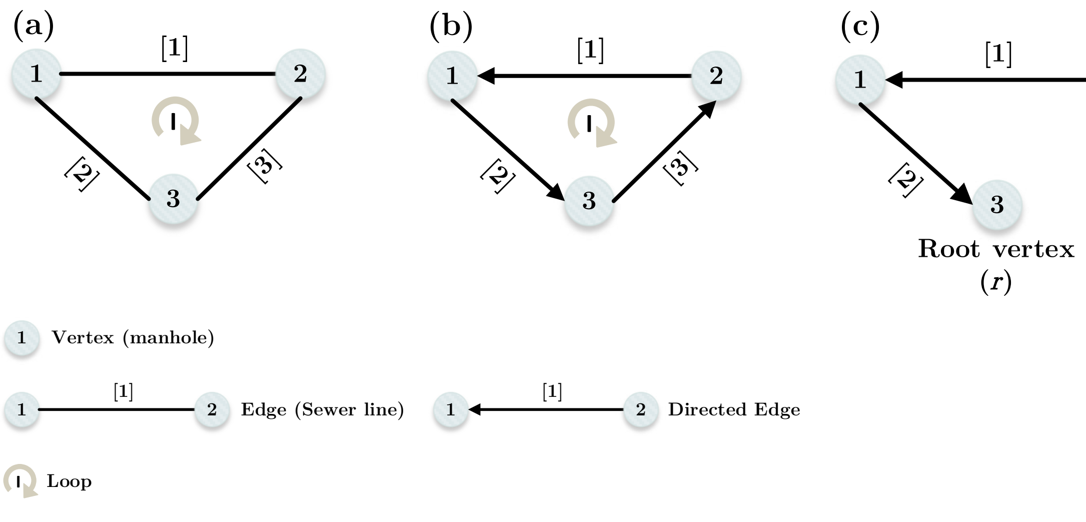


Figure 2.1: *Sample graphs: (a) undirected graph; (b) cyclic digraph; (c) acyclic digraph*

With the definitions mentioned above, the layout of an urban drainage network is a sub-graph derived from a pre-defined base graph. In a base graph, all drainage possibilities are involved in a way that manholes (vertices) and sewers (edges) establish a connected cyclic graph. Concerning the street alignments, topology, barriers, watercourses, location of the outlet and existing sewers in the area under design, an undirected base graph can be drawn. Each manhole within the base graph is potentially possible to be connected to its adjacent manholes toward which the flow is directed.

Sewer Layout Constraints

For generating a feasible layout from a base graph, the following primary constraints need to be met [52] :

1. No cycle is accepted. Therefore, the layout is a tree,
2. All manholes (vertices) must be included in the tree so that the layout is a spanning tree,
3. All sewers (edges) must be involved in the tree since each of them drains a particular area,
4. There is an outlet (root) in each isolated part of the system toward which each spanning tree must be directed, and
5. Several sewers can flow into a manhole. However, except for the root node, exactly one sewer leaves every non-root manhole in the direction of the root.

As a result, a possible layout is an arborescence spanning tree with a root, that finally discharges to a wastewater treatment plant, a main sewer collection line, or a water body. In cases with more than one outlet (decentralized networks), nr roots, a forest of nr arborescences represents the layout.

2.3 Hydrological-Hydraulic modeling of the system

As mentioned before, the next step after designing the layout of the system is to size the components of it in a way that satisfies all technical and hydraulic constraints. To this aim, it is crucial to model flow in the UDS.

Setting up models is expensive, and without clear modeling objectives, time and financial resources can be wasted in doing unnecessary tasks [56]. Therefore, system authorities have to be careful about defining the aims of each modeling implementation so that the project is carried out at the right level of detail and expense [4]. For UDSs, there are at least four primary reasons to build a hydraulic simulation model [4, 56]:

1. **Overall planning (coarse drainage area planning or master planning)**: the model is used for master planning over a large catchment or to priorities several UDSs scenarios. The resolution of these types of models is low and are not suitable for the detailed representation of flooding problems. The simplification level for a coarse drainage area planning model is about 1.0 to 3.0 ha/pipe, 100-200m for the core areas.
2. **Detailed drainage area planning**: the model would be used for asset management planning at a higher level of detail, including the evaluation of an investment in specific construction projects, identification of parts of the system that need particular attention, and the confirmation of the value of a given network. These types of models include major core sewers (simplification level: 0.5 to 2.0 ha/pipe, 10-100m for core areas), simplified peripheral areas, all ancillary structures, and detailed modeling of the area with known flooding and surcharging.
3. **Detailed design**: the model would be employed for detailed investigations and detailed design of new components of a system. These models typically include all relevant pipes, manholes and ancillary structures.
4. **Sewer quality**: such models are necessary when there are concerns about the environmental impact on receiving waters. Generally, these models need a high degree of detail.

The frameworks that are presented within this thesis are appropriate for overall planning and detailed drainage area planning. Among existing hydrological-hydraulic modeling systems such as InfoWorks, MIKE URBAN, MUSIC, MOUSE, MIKE 21, The Storm Water Management Model (SWMM) is selected in this thesis. The reasons for this choice are explained in the following paragraphs.

SWMM is a dynamic rainfall-runoff simulation model that can be employed for a single event or long-term simulation of runoff quality and quantity from predominantly urban areas. The runoff component of SWMM functions on a collection of sub-catchment areas that receive precipitation and generate runoff and pollutant loads [59]. The routing module of SWMM conveys this runoff through a system of pipes, channels, storage/treatment facilities, pumps, GBIs and regulators. SWMM tracks the quality and quantity of runoff generated within each sub-catchment, and the flow specifications such as flow rate, flow depth, and quality of water in each pipe during a simulation period comprised of multiple time steps [59, 60]. Besides the capacities as mentioned above, the most important reason for choosing it for the aim of this thesis is that SWMM is a free open source model that makes it possible to easily be linked with external layout modeling codes and any optimization engines.

Figure 2.2 schematically depicts the processes modeled by SWMM to simulate the input of the system (rain or wastewater) and the hydraulic response of the sewer system in terms of flow-rate and depth. The blue blocks are used within this thesis and are described briefly in the following paragraphs. It worth mentioning that each block contains different simulation options and sub-processes. Here, only the processes that are used within this thesis are presented. The material of the following sections is mainly adopted from [61] for the rainfall-runoff modeling [59] for the flow routing through pipes in the network, and [62] for modeling the GBIs.

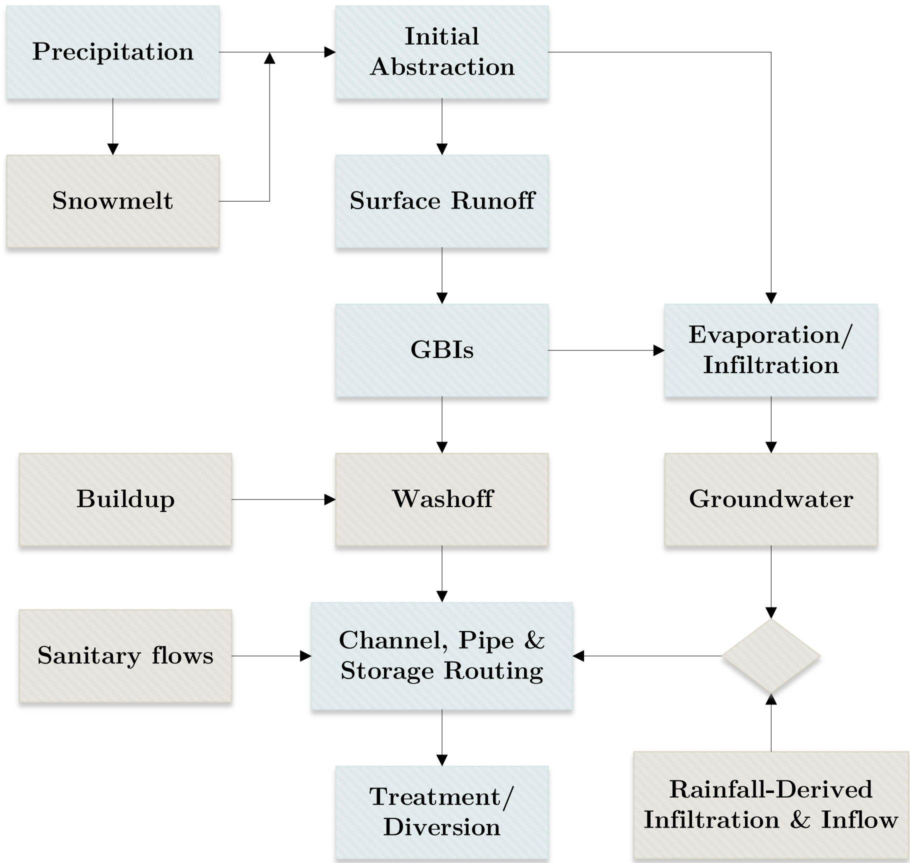


Figure 2.2: *Processes modeled by SWMM* [59]

2.3.1 Rainfall-runoff modeling

Many SWMM analyses rely upon rainfall data provided by the user, based on measurements made at the closest rain gauges to the catchment, or on an assumed design storm. Both real design storms that are obtained from actual measurements and synthetic ones that are derived from an assumed duration and temporal distribution can be employed. Since SWMM does not provide synthetic design storms automatically, they must be constructed as described in the following paragraph.

Synthetic design storms

The probabilistic relationship between average rainfall intensity, duration and return period are often depicted in graphical form and referred to as intensity-duration-frequency (IDF) curves [63]. Figure 2.3 shows the IDF curves for the city of Ahvaz in Iran. It can be observed that the intensity decreases with the duration of the rainfall and increases when the return period is increased. Constructing IDF curves is done using a so-called frequency analysis [4].

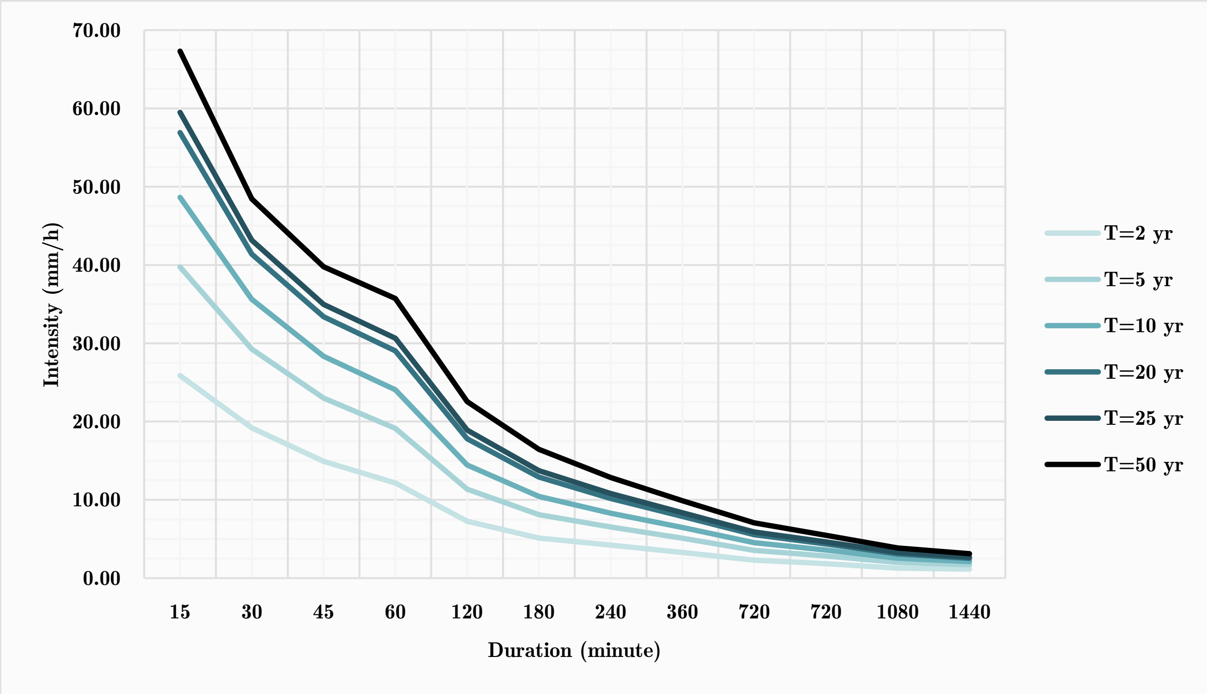


Figure 2.3: *IDF curves for the city of Ahvaz, Iran*

Various standard dimensionless or semi-dimensionless hyetographs (a graphical representation of the distribution of rainfall intensity over time) can explain the temporal distribution of rainfall over the storm duration. There are several methods to constructs these standard hyetographs using available IDF curves [63]:

1. Soil conservation service method SCS
2. Yen and Chow method
3. Huff method
4. Chicago method
5. Synthetic block hyetograph method

Details about each method are given in in Akan and Houghtalen (2003) [63]. In this thesis, the synthetic block hyetograph method is used to construct synthetic design storms as suggested in the regional guidance manual. Figure 2.4 depicts these design storms derived from the presented IDF curves in Figure 2.3 using this method for design storms with return periods 2, 5, 10, 20, 25 years and a duration of six hours. These synthetic design storms have been used within this thesis for optimization and design.

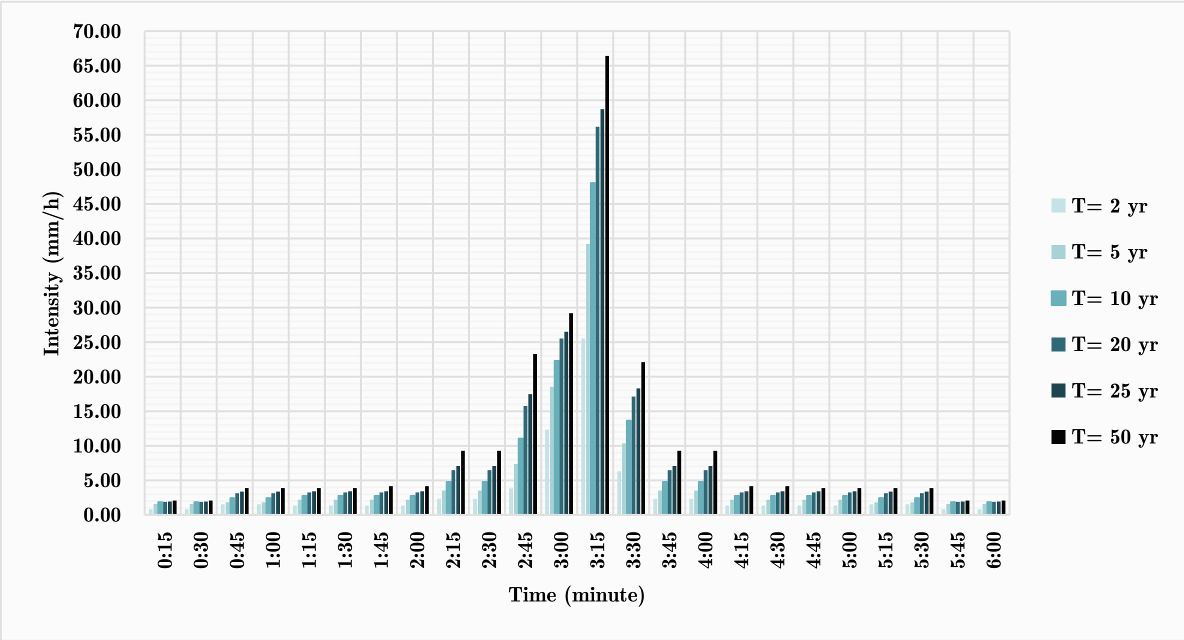


Figure 2.4: *Synthetic design storms with different return periods for the city of Ahvaz.*

Surface Runoff

This section explains how SWMM transforms precipitation excess (precipitation minus losses such as infiltration, evaporation, and initial abstraction) into surface runoff (overland flow). After calculating the losses from the catchment, as explained in the next section, the effective rainfall hyetograph can be converted into a surface runoff hydrograph.

SWMM adopts a nonlinear reservoir model to assess surface runoff produced by rainfall over a sub-catchment, as depicted in Figure 2.5. SWMM conceptualizes a sub-catchment as a rectangular surface that has a uniform slope S and a width W that drains to a single outlet. In this demonstration, the sub-catchment encounters inflow from precipitation and losses from evaporation and infiltration. The remaining water ponds above the sub-catchment surface to a depth d_s. Ponded water above the depression storage depth can become runoff outflow. Depression storage estimates the initial rainfall abstractions such as surface ponding, surface wetting, and an interception by flat roofs and vegetation.

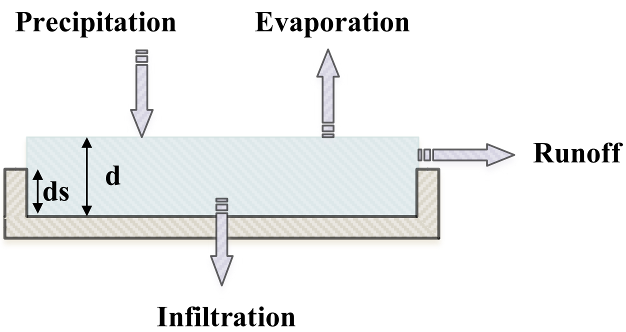


Figure 2.5: *Nonlinear reservoir model of a sub-catchments* [61].

From the conservation of mass, the net change in depth d per unit of time t is the difference between inflow and outflow rates over the sub-catchment [61]:

$$\frac{\partial d}{\partial t} = i - e - f - q \tag{2.1}$$

Where
$i =$ rate of rainfall + snowmelt (m/s)
$e =$ surface evaporation rate (m/s
$f =$ infiltration rate (m/s)
$q =$ runoff rate (m/s)
In this formulation, the fluxes i, e, f, and q are expressed as flow rates per unit area.

SWMM uses the Manning equation to express the runoff's rate q, assuming that flow across the sub-catchments surface acts as if it were uniform flow within a rectangular channel of width W, height $d - ds$, and slope S as presented in Equation 2.2.

$$q = \frac{WS^{1/2}}{An}(d - d_s)^{5/3} \tag{2.2}$$

where
$n =$ surface roughness coefficient
Substituting Equation 2.2 into Equation 2.1 results in:

$$\frac{\partial d}{\partial s} = i - e - f - \alpha(d - d_s)^{5/3} \tag{2.3}$$

where α is defined as:

$$\alpha = \frac{WS^{1/2}}{An} \tag{2.4}$$

Equation 2.3 is an ordinary nonlinear differential equation. For known values of i, e, f, d_s and α it can be solved numerically over each time step for ponded depth d. Once d is known, values of the runoff rate q can be found from Equation 2.2.

Rossman and Huber (2016) [61] provides some guidelines and suggestions for estimating sub-catchment parameters including area, imperviousness, width, slope, Manning's roughness coefficient and depression storage.

Infiltration

Infiltration is defined as the process of penetrating the rainfall into the ground surface and filling the pores of the underlying soil (Akan and Houghtalen, 2003). For calculating infiltration f in equation 2-1, SWMM is equipped with four methods: (1) Horton's method, (2) modified Horton method, (3) the Green-Ampt method, and (4) the Curve Number method. In this thesis, Horton's method is used for modeling the infiltration in SWMM.

Horton 1941 [64] suggested an exponential equation to predict the reduction in infiltration capacity over time as follow:

$$f_p = f_\infty + (f_0 - f_\infty)e^{-k_d t} \quad (2.5)$$

where:

f_p	=	infiltration capacity into soil (m/s)
f_∞	=	minimum or equilibrium value of f_p (at t=∞)(m/s)
f_0	=	maximum or initial value of f_p (at t=0)(m/s)
t	=	time from the beginning of storm(sec)
k_d	=	decay coefficient (1/sec)

2.3.2 Flow routing through pipes

Analysis Methods

The hydraulics of unsteady non-uniform flow is expressed in SWMM by a pair of partial differential equations for conservation of mass and momentum (see Equations 2.6 and 2.7) known as the St. Venant equations. The simultaneous solution of these equations for each pipe, coupled with conservation of volume at each node, gives information on the spatial and temporal fluctuation of water levels and flow rates throughout the pipe network.

SWMM offers the user three primary alternative methods for routing the flow through pipes, (1) steady flow analysis, (2) kinematic wave analysis, or (3) dynamic wave flow analysis. The first option is not based on the St. Venant equations.

The steady flow analysis option assumes that, within each computational time step, flow is uniform and steady. This method translates inflow hydrographs at the upstream end of a pipe to its downstream end, without considering any change in shape or any delay. Here, to explain the relation between flow rate to flow depth, the Manning equation is utilized. Its limitations are similar to those of the kinematic wave method (see below). Since it neglects the dynamics of free surface wave propagation, it is only suitable for coarse preliminary analysis of long-term continuous simulations.

Kinematic wave analysis solves the continuity equation along with a simplified form of the momentum equation in each pipe. This simplified approach cannot consider entrance/exit losses, backwater effects, flow reversal, or pressurized flow. Its application must be restricted to steeply sloped pipes with the shallow flow and high velocity.

Dynamic wave analysis solves the complete form of the St. Venant equations. Consequently, it provides theoretically the most accurate results. Unlike the first two simulation methods, this method can account for entrance/exit losses, flow reversal, channel storage, backwater effects and pressurized flow. Besides, it can be implemented for any general network layout, even those with several outlets and loops. This ability is the most crucial reason to choose the dynamic wave method for all analyses within this thesis, as layouts with multiple outlets cannot be modeled in the other approaches. However, all these advantages come at the price of having a much larger computational burden in comparison with steady-state or kinematic wave analysis. In the following paragraphs, the governing equations of dynamic wave analysis are given.

Governing equations

The St. Venant equations can be expressed as [59]:

$$\frac{\partial A}{\partial x}+\frac{\partial (Q)}{\partial t}=0 \qquad \text{Continuity} \qquad (2.6)$$

$$\frac{\partial Q}{\partial t}+\frac{\partial (Q^2/A)}{\partial x}+gA\frac{\partial H}{\partial x}+gAS_f=0 \qquad \text{Momentum} \qquad (2.7)$$

where:

$x =$ distance (m)
$t =$ time (s)
$A =$ flow cros-sectional area (m^2)
$Q =$ flow rate (m^3/s)
$H =$ hydraulic head of water in the conduit (Z+ Y) (m)
$Z =$ conduit invert elevation (m)
$Y =$ conduit water depth (m)
$S_f =$ friction slope (head loss per unit length)
$g =$ acceleration of gravity (m/s^2)

The main assumptions to derive these equations are:

1. The fluid is incompressible
2. flow is one-dimensional,
3. the pressure is hydrostatic,
4. the cosine of the channel bed slope angle is close to unity,
5. boundary friction can be expressed as for steady flow [59].

The Manning equation can express the friction slope S_f to model steady uniform flow:

$$S_f=n^2\frac{Q|U|}{AR^{4/3}} \qquad (2.8)$$

In these equations, the flow area A is a known function of water depth Y that can be obtained from the head H. Therefore, flow rate Q and head H are the dependent variables that are functions of time t and distance x. A set of boundary conditions for Q and H at $x=0$ and $x=l$ at all times, as well as initial conditions at time 0, is needed to solve these equations over a single pipe of length L.

The continuity equation 2.6 can be merged with the momentum equation 2.7 to give the following form of the momentum equation for a pipe:

$$\frac{\partial Q}{\partial t}=2U\frac{\partial A}{\partial t}+U^2\frac{\partial A}{\partial x}-gA\frac{\partial H}{\partial x}-gAS_f \qquad (2.9)$$

Besides equation 2.9 that is used to calculate the time trajectory of flow Q in a pipe, an additional equation is required to do the same for heads H. SWMM solves this issue by providing a continuity relationship at junction nodes that connect pipes within an urban drainage network. It is supposed that a continuous water surface exists between the water elevation at a node and in the pipes that enter and leave it. Each node assembly, in SWMM representation, is the summation of each node itself and half the length of each link connected to it. Conservation of flow for the assembly needs that the change in volume over time equals the difference between inflow and outflow, as presented in Equation 2.9 [59].

$$\frac{\partial V}{\partial t} = \frac{\partial V}{\partial H}\frac{\partial H}{\partial t} = A_s \frac{\partial H}{\partial t} = \sum Q \tag{2.10}$$

$$A_s = A_{SN} + \sum A_{SL} \tag{2.11}$$

Finally, combining 2.10 and 2.11 constructs Equation 2.12.

$$\frac{\partial H}{\partial t} = \frac{\sum Q}{A_{SN} + \sum A_{SL}} \tag{2.12}$$

where:

$V =$ node assembly volume (m^3)
$A_s =$ node assembly surface area (m^2)
$A_{SN} =$ node's storage surface area (m^2)
$A_{SL} =$ total area contributed by the links connected to node assembly (m^2)
$\sum Q =$ net flow into the node assembly (inflow – outflow) (m^3/s)

The flow depth at the end of a pipe connected to a node can be estimated as the difference between the head at the node and the invert elevation of the pipe. The node and link surface areas are calculated as functions of their respective flow depths.

Equations 2.9 and 2.12 provide a coupled set of partial differential equations that solve for flow Q in the pipes and head H at the nodes of the urban drainage network. As there is no analytical method to solve them, a numerical solution method must be employed instead. The solution method is given in [59].

Sewer flooding

Sewer flooding happens when runoff exceeds the conveyance capacity of the UDS. As a consequence, exceedance flow is generated on the urban surface. One of the essential roles of sewer flow models is to represent the performance of the system during threats such as extreme rainfall events.

In general, four types of models can be found in the literature to model this phenomenon [4]:

1. **Virtual flood cones:** In this method, the flooding extent is estimated by assuming a virtual cone or reservoir on top of each manhole (node). This reservoir can temporarily store the floodwater. The water can flow back to the drainage system as long as the capacity allows, or it can be removed from the system entirely. This approach is computationally inexpensive and provides a reliable indication of surface flood volume; however, it only gives a rough estimate of flow depth.
2. **Rapid flood spreading models:** These methods use the volume of flow produced by the pipe flow model and try to distribute it more realistically over the catchment surface. These approaches are computationally inexpensive as well and are relatively easy to employ; however, they do not represent any flow movement and have no time component.

3. **1D–1D models:** In this concept, the urban area is treated as a network of open channels and ponds connected to the network of pipes. The hydraulics of open channel system is solved with an approach similar to pipe hydraulics. These models can specify the detailed overland flow routes to some extent; however, they cannot capture the interactions between above- and below-ground flow.
4. **1D–2D models:** These models couple 1D pipe flow models with 2D surface flow models. The 2D flood routing model solves a version of the Saint-Venant equations as well. They are considered to be the most accurate representations of urban surface flooding. However, achieving this accuracy comes at a high computational burden both in terms of time and data demands. Such models could be used for detailed design.

SWMM uses the first approach, virtual flood cones, for flood modeling, which is appropriate for overall planning using highly time-consuming simulation-optimization methods.

In SWMM representation, when a node is allowed to pond, flooded water stays in the system. Therefore, the ponded depth above the node increases during periods of flow excess and decreases during periods of flow deficit. A node with a large ponded area encounters smaller changes in ponded depth for a given flow surplus than one with a small ponded area. Selection of which nodes can pond and their corresponding ponded areas depends on local topography, typically happening along with flat districts or at sag spots of the drainage system.

2.3.3 Modelling GBIs

GBIs are distributed or decentralized practices that aim to control runoff at the source. To this aim, typically, they modify the landscape or surface located on or near to impervious areas where most of the runoff is generated.

As mentioned before, SWMM can model several types of GBIs such as bio-retention cells, rain gardens, green roofs, infiltration trenches, permeable pavement, and vegetative swales. The following paragraphs provide the governing equations and the modeling approach in SWMM for two types of GBIs; infiltration trenches and rain barrels, are given in short. These two types of GBIs are used in this thesis. The reasons for this choice are explained in Chapter 4. The details about modeling other types of GBIs can be found in [62].

Infiltration Trenches

Infiltration trenches are narrow trenches filled with gravel that capture runoff from upstream connected impervious areas. Their storage volume provides extra time for captured runoff to infiltrate into the native soil below [62].

Conceptually, an infiltration trench can be represented by a surface and a storage layer, as shown in Figure 2.6. The surface layer collects both direct rainfall and runoff captured from other areas. It loses water through evapotranspiration, infiltration into the soil layer below it and by any surface runoff that might happen. The storage layer consists of coarse crushed stone or gravel. It permits infiltration from the surface zone and loses water by infiltration into the underlying natural soil and by drainage through a perforated pipe underdrain system if available [62].

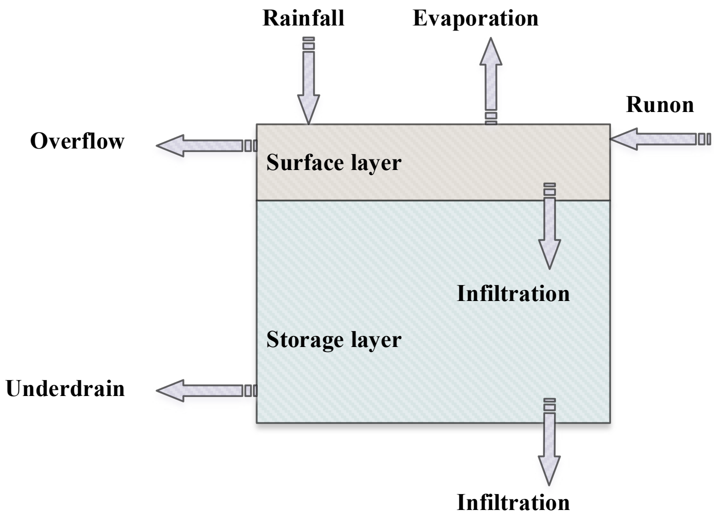


Figure 2.6: *Conceptual model of a typical infiltration trench*

Governing equations

Infiltration trenches are modeled in SWMM by solving a set of flow continuity equations. Each equation describes the variation in water content in a distinct layer over time as the difference between the inflow and the outflow water flux rates that the layer observes, expressed as volume per unit area per unit time. These equations are written as follows:

$$\frac{\partial d_1}{\partial t} = i + q_0 - e_1 - f_1 - q_1 \qquad \text{Surface Layer} \qquad (2.13)$$

$$\varphi_2 \frac{\partial d_2}{\partial t} = f_1 - e_2 - f_2 - q_2 \qquad \text{Storage Layer} \qquad (2.14)$$

Where:

$d_1 =$ depth of water stored on the surface (m)
$d_2 =$ depth of water in the storage layer (m)
$i =$ precipitation rate falling directly on the surface layer (m/s)
$q_0 =$ inflow to the surface layer from captured runoff (m/s)
$q_1 =$ surface layer overflow rate (m/s)
$q_2 =$ storage layer underdrain outflow rate (m/s)
$e_1 =$ surface evapotranspiration rate (m/s)
$e_2 =$ storage layer evapotranspiration rate (m/s)
$f_1 =$ infiltration rate of surface water into the soil layer (m/s)
$f_2 =$ exfiltration rate of water from the storage layer into native soil (m/s)
$\varphi_2 =$ the void fraction of the storage layer (void volume / total volume)

SWMM solves this set of coupled equations using numerical methods at each runoff time step. Solving these equations determines how an inflow hydrograph to the infiltration trench is transformed into hydrographs for surface runoff, underdrain outflow, and exfiltration into the surrounding native soil. Details about how to calculate each of the flux terms in Equation 2.13 and 2.14 are given in [62].

Rain Barrels

Rain Barrels are containers that save roof runoff during rain events and can release or re-use the collected water during dry periods [62].

SWMM models a rain barrel as only a completely void storage layer with a drain valve placed above an impermeable bottom. Hence, just a single continuity equation is required:

$$\frac{\partial d_2}{\partial t} = f_1 - q_1 - q_2 \qquad \text{Storage Layer} \qquad (2.15)$$

where

$f_1 =$ surface inflow captured by the barrel (m/s)
$q_1 =$ surface layer overflow rate (m/s)
$q_2 =$ storage layer underdrain outflow rate (m/s)

2.4 Decision making with optimization techniques

This section presents an introduction to optimization techniques and their applications for decision making in urban water management problems. The main contents are adopted from [57, 65].

Nowadays, many urban water management decisions try to find a preferred option from a list of options, such as a range of designs, planning, operational, management and policy scenarios. Environmental models like urban drainage models are utilized broadly to aid these decision-making procedures by assessing the performance of different alternatives.

To aid decision making using simulation models, the inputs of models must correspond to the intended decision choices. Besides, the model outputs must correspond to the outcomes of interest. Here, the possible decision choices (e.g., pipes diameters) are called ***decision variables***, and the outcomes of interest are usually referred to as ***objectives***. The problem objectives are the values that must be minimized (e.g., construction costs) or maximized (e.g., resilience). In this context, ***constraints*** are the allowable values of decision variables (e.g., the maximum pipe diameter available in the market) and objectives (maximum allowable flow velocity). The values of choice for all decision variables is called a ***solution*** to the defined problem.

Identifying the ***optimal solution***, the best solution in terms of the objective function(s), might be challenging in the real-world, as environmental problems are generally complicated, and the number of feasible options is usually notable. In general, three approaches may be used to find the best solutions:

1. **Trial-and-error:** First, based on experience and knowledge of the analyst, a plausible solution is selected. Then, the performance of the generated solution in terms of objective function values and constraint violation is assessed typically using models. Next, an alternative, optimistically better, solution is generated based on the performance of the previous (initial) solution and analyst experience and knowledge. The performance of the new solution is evaluated as well. These steps are iterated until no further improvement in objective function values can be achieved, or the analyst is satisfied with the final solution. This approach is applicable only when the analyst has a robust understanding of the problem and simulation models, and the number of decisions and their possible range is limited. As for most urban water problems, the search space is enormous and this approach only can cover a tiny fraction of the total search space; finding optimal or even near-optimal solutions is highly unlikely.
2. **Full enumeration,** in which all solution alternatives first are generated and then are evaluated using simulation models. The best of the existing solutions is finally chosen. This approach can guarantee to reach the global optimum; however, it is not practical for real-world problems where the number of feasible solutions is incredibly high and the full enumeration of all possible solutions might be computationally impossible.
3. **Formal optimization** includes a range of algorithms. These range from simple mathematical-based optimization methods such as linear programming to intelligent metaheuristic methods such as genetic algorithms. These methods are developed and widely used for the optimization of complicated engineering problems. They are known for their ability to find globally optimal or near-optimal solutions (with a dramatically improved solution quality compared with the results of trial-and-error approach) in plausible computation time.

From the above introduction, it can be concluded that, for real-world UDSs optimization with many decision variables, constraints and often conflicting objectives, the only promising approach is formal optimization. The choice of the optimization engine from the long list of the available methods depends highly on the type of the problem (e.g., design, rehabilitation or operation problem), number and type of decisions, number and nature of objectives and constraints. When an analytical representation of the single objective function is available, one of the following approaches might be employed to solve it [66]:

- **Linear Programming (LP):** This optimization technique is used when objective function and constraints are mathematically represented by linear relationships. Some common algorithms for LP are the *Simplex algorithm*, *interior-point methods*, *Column generation*.
- **Integer Linear Programming (ILP):** In LP, when all of the decision variables are restricted to be integers such as the size of pipe diameter, the problem is known as an ILP. Binary integer programming (BIP) is a particular instance of ILP, where all decision variables are obligated to be 0 or 1. If decision variables are a mix of integer and non-integer values (e.g., size of pipes and their slopes), the problem is called a mixed-integer linear programming (MILP) problem. These types of optimization problems might be solved using algorithms such as *cutting-plane method*, *branch and bound*, *branch and cut* and *branch and price*.
- **Dynamic Programming (DP):** This technique simplifies a complex problem by splitting it into more manageable sub-problems in a recursive manner. A problem that can be determined optimally by splitting it into sub-problems and then recursively obtaining the optimal solutions to the sub-problems is said to have an optimal substructure. In other words, the combination of the optimal solutions of these sub-problems can result in an optimal solution to the original complicated problem [67]. This approach has applications in a wide range of disciplines including graph theory.
- **Non-Linear Programming (NLP):** The difference between NLP and LP is that in NLP there is at least one non-linear objective function or constraint. For non-linear problems that have a global optimum but no local optima (such as convex or concave problems), gradient-based optimization methods such as *gradient descent* or the *Newton–Raphson* method are commonly used (if all objective functions and constraints are differentiable with respect to the decision variables). For these methods, information about the gradient of the objective function is needed to determine the direction of the search. However, the gradient (derivative) information on water resources or environmental models is not typically available. As a remedy, the derivative of the objective function can be approximated numerically. This approach raises the computational effort; as many more objective function evaluations are required. Besides, in many real-world problems, the objective functions are not differentiable, for example, because of discontinuity in some areas of the search space. Moreover, many optimization problems are multi-modal or non-convex, which means many local optima might be available in the feasible search space. Escaping local optima and searching for the global optimum can be possible either by a modification in gradient-based algorithms or doing the optimization many times with multiple random initial solutions. Both of these approaches increase the computation burden, particularly for real-world, large-scale problems.

Most of the optimization problems in water and environmental engineering are non-linear, multi-modal, large scale and most functions might be not known analytically (so gradients cannot be calculated). Therefore, they cannot be tackled by any of the above-mentioned methods. To handle such a complicated optimization problem, ***global optimization*** methods (also often referred to as ***metaheuristics***) could be employed.

Global Optimization by metaheuristics

In the last two decades, various intelligent meta-heuristic methods have been developed and widely applied to the optimization of complex engineering problems. Although finding the global optimum cannot be guaranteed through most of these methods, it has been shown that they are very promising in approximating the global optimum in multimodal search spaces in a plausible time. These easy-to-understand-and-implement techniques can be used to solve multi-objective mixed-integer nonlinear problems. Besides, meta-heuristics can straightforwardly handle constrained problems using penalty functions. Using penalty functions, a constrained optimization problem can be transformed into an unconstrained problem by adding a penalty value to the objective function. The amount of penalty value depends on the degree of constraint violation of a candidate solution.

Meta-heuristics can be classified in many categories, such as deterministic and stochastic search, or population-based and single-solution based search. A meta-heuristic method is ***stochastic*** (e.g., *Simulated Annealing*, *Genetic Algorithm*), when it uses any randomized decisions in its structure. On the other hand, a meta-heuristic method is ***deterministic*** (e.g., *Tabu Search*) when there is no randomness in it. Single-solution based meta-heuristics (e.g., *Tabu Search, Simulated Annealing)* works only based on one possible solution at the same time. Population-based meta-heuristics (e.g., *Particle Swarm Optimization*, *Genetic Algorithms, Ant Colony Optimization*), on the other hand, investigate the search space concurrently with many solution candidates. These candidates interact with each other and use swarm intelligence to determine the optimal solution.

The nature of the problem at hand can determine the choice of the meta-heuristics. This includes the number and types of decision variables (e.g., binary, integer, real, or mixed-integer variables), number of objectives (single or multi-objective) and whether the problem is constrained or unconstrained. Generally, the best results might be obtained when the analyst tune the meta-heuristics parameters well based on his or her experience, knowledge and intuition about the problem.

In the next section, the problem of designing UDS is formulated as an optimization problem. Then, based on the problem characteristics, proper optimization methods from the available techniques mentioned in the current section are chosen and described in more detail.

2.4.1 Design of UDSs as an optimization problem

Mathematically, optimization problems can be expressed as a maximization problem of an objective function, subject to inequality or equality constraints, as follows [65]:

$$\text{maximize or minimize } (f_1(d), f_2(d), \dots, f_k(d)) \tag{2.16}$$

subject to

$$\begin{aligned} g_i(d) &\le 0, & i &= 1,2,\dots,m \\ d_{jl} \le d_j &\le d_{ju} & j &= 1,2,\dots,n \end{aligned}$$

where d is the vector of decision variables, d_j indicates the j^{th} value in this vector, and d_{jl} and d_{ju} are lower and upper bounds on the decision variables, respectively; f_k is the k^{th} objective function and g_i is the i^{th} constraint function of vector d. In this formulation, a problem is called a ***single-objective optimization*** problem when $k = 1$ and it is called a ***multi-objective optimization*** (MOO) problem when $k > 1$.

As for the UDSs depending on the aim of modeling and optimization (design, rehabilitation or operation), decision variables can include layout configuration, hydraulic specification of different components of the system including their type, size and location, pump scheduling program, controlled outflows from combined sewer overflow tanks, etc. Furthermore, objective functions can be minimizing construction costs, total damage due to urban flooding, energy consumption, total discharged pollutions, or maximizing system reliability, resilience, sustainability, etc. Section 2-1 to 2-3 presented some technical and hydraulic constraints related to the design of UDSs. In some cases, it is possible to impose some constraints as objective functions (e.g., maximum allowable discharged pollutions) in the problem formulation and vice versa.

As mentioned in section 2-1, the optimal design of UDSs as pursued within this thesis needs to optimize the design of the network layout and the size of the network's components. These two sub-problems are very different from each other mathematically, nevertheless, none of them has an analytical representation. The sub-problem of designing the layout belongs to a challenging class of combinatorial optimization in graph theory. The second sub-problem, sizing the components, is a nonlinear discrete program that also can be considered as a hierarchy decision-making problem. Both of these problems are mixed-integer nonlinear and highly constrained, and they could be extremely multimodal, depending on the formulation of the objective functions. Therefore, only global optimization methods (metaheuristics) can be employed here to find optimum solutions.

Two single-objective optimization engines, namely *Tabu Search* (TS) and *Genetic Algorithm* (GA) and one multi-objective optimization engine known as *Borg Multi-objective evolutionary algorithm* (Borg MOEA), have been employed within this thesis for optimization purposes. TS has been selected for the optimization task in Chapter 3 because of its excellent performance for problems with integer decision variables [52]. Chapter 4 enjoys a binary formulation of GA that reduces the search space, as is explained in that chapter. For the aim of MOO in Chapter 5, the Borg MOEA has been selected because of its demonstrated robust performance to handle extremely multi-modal problems with a high number of decision variables and objectives [68]. Finally, a *Multi-criteria decision analysis* (MCDA) technique named TOPSIS is employed in Chapter 5 to rank the non-dominated solutions found by MOO by including a wide range of indicators.

Tabu Search (TS)

Hertz and Werra (1987) [69] and Glover (1989) [70], firstly introduced and formalized the TS metaheuristic optimization method. Glover (1990 and 1995) [71, 72], then developed TS and Hertz and Werra (1990) [73] popularized it. So far, this technique has been applied to solve diverse combinatorial optimization problems in various fields of engineering and economics. The foremost advantage of TS compared to other metaheuristics is that TS is a deterministic search, which means there is no randomness in it. As a result, TS is commonly observed to be computationally efficient.

TS is based on the local search method utilized for mathematical optimization but improves the performance of it by relaxing its basic rule. Simply phrased, TS begins the investigation of a search space with an arbitrary starting point. In the vicinity of the current point, the best solution is explored considering the optimization problem objective function. Whether the best neighbor solution is better than the current solution or not, it is chosen as the new solution and the search proceeds. This feature, accepting even worse neighbors and continuing the exploration, is the key equipment of TS that enables it to escape local optima in comparison with local search algorithms. However, this feature might significantly raise the number of objective function evaluations and consequently increases the total time it takes to complete the exploration of the search space. Besides, the search probably becomes trapped in a loop of succeeding solutions that periodically drives to identical results.

To handle these concerns, TS systematically utilizes memory of the search that describes the visited solutions, to determine new solutions and search directions. To this aim, the best solution at each iteration or the move toward it is stored in a Tabu list. Users define the length of the Tabu list as a parameter in TS upon which a long or short memory search is formed. Throughout the search, TS is prohibited to pick solutions from the Tabu list, even if those points are superior to other neighbors. With this approach, no visited solution is visited again. As a result, the exploration does not fall in a loop of solutions anymore.

Besides this feature, there are some other innovations in TS that refine the search's efficiency and accuracy namely *diversification* and *intensification*. Diversification makes it possible to escape local optima by diversifying the search engine in the problem's search space. Intensification, on the other hand, improves the accuracy of the optimum solution by biasing the exploration towards promising areas of the search space.

Herein, a TS algorithm is presented as follows that can be coupled to a simulation-optimization model. This algorithm adopted from [52, 57]

1. An arbitrary initial design alternative d is generated with respect to the decision variable bounds. This solution is termed K, and let $K^* = K$, where K^* is the best solution so far explored.
2. Set the cycle number $j = 1$.
3. Set the iteration number $i = 1$.
4. Solution K is sent to the Tabu list T with a user-specified length $|T|$.
5. For solution K, a neighborhood zone $N(K)$ in the problem's decision space is produced. The formation and generation strategy of $N(K)$ play a significant role in the performance and accuracy of a TS run. For the sewer design problem, the univariate search direction method has been found proficient and easy to use for generating the neighborhood [52].
6. The feasible neighborhood, named V^*, is achieved by removing the Tabu solutions from the generated neighbors such that $V^* = N(K) - N(K) \cap T$. For all points in V^*, the objective function is evaluated. With no comparison with the current solution, the best solution in V^* is found and termed as U. Set $K = U$, and if the objective function $C(K) < C(K^*)$ (for a minimization problem), let $K^* = K$. Also, let $i = i + 1$ and go to step 4 until all directions are once sought.
7. Set the cycle number $j = j + 1$. At this point, TS has partially sought the decision space in all directions. Now, it is said that a search cycle has been completed.
8. If the user-defined stopping criteria are met the search is terminated. Otherwise, go back to step 3.

Genetic Algorithm (GA)

GAs are based on the principles of natural genetics and natural selection. The GAs are inspired by Darwin's evolution theory and first introduced by Holland (1975) [74]. Afterward, it was extended by Goldberg and Holland (1988) [75] and broadly used to solve many complicated engineering problems. GAs are stochastic metaheuristic algorithms that belong to the larger class of evolutionary algorithms (EA). In the following, a standard continuous GA is presented.

1. An initial population with NP chromosomes is randomly generated considering the decision variables' bounds. Each chromosome contains a design candidate d_j and, each design variable inside is termed as a gene.
2. The objective function (fitness) for all chromosomes ($j = 1 \ldots NP$) is evaluated.
3. A tournament selection method is applied to select the parents in such a way that, for each parent depending on the tournament selection method, k ($k \geq 2$) chromosomes d_1

to d_k are randomly picked up from the population. d_j wins the tournament if it has better fitness than others in the tournament. The number of parents is a user-defined parameter generally considered to be half of the initial population size ($\frac{NP}{2}$). After selecting all parents, they are carried to the mating pool to generate new offsprings.

4. A crossover method such as the blend crossover method [76] is applied to each couple in the mating pool to produce two children. When the crossover operator is applied to all couples, the population of children with size NP is created.
5. A few genes in the new population are mutated based on a user-defined mutation ratio.
6. The fitness of the children's population is evaluated.
7. The new and old populations are merged to construct a population with $2NP$ size. The merged population is sorted according to their fitness. The topmost NP chromosomes with better fitness form the new generation are selected. Since the new generation is derived from the combination of parents and children, the elitism is systematically conserved in this algorithm.
8. The algorithm checks whether the convergence criteria are met. If no further enhancement was observed in the results or the number of generations exceeds a user-defined value, the optimization is terminated otherwise, the algorithm with a new population is iterated from step 2.

Borg MOEA

The main difference between single-objective EAs like GA and multi-objective EAs (MOEAs) such as Borg MOEA is in their approach to evaluate the fitness of a solution. MOEAs require a more sophisticated fitness assignment scheme than simply evaluating a single objective function to bring together and conjoin several often conflicting objective functions. In general, fitness assignment involves ranking and selecting high-quality solutions during the exploration procedure by translating a vector of objective function values into a scalar-valued fitness. Most fitness assignment strategies are based on the dominance concept that rank candidate solutions considering their dominance strength in objective space. The result of an MOEA is a Pareto-front that reveals the trade-off between the conflicting objectives. It provides the user, also, with the final set of decision variables [65].

Most parts of the remainder of this section are obtained from [68]. The Borg MOEA employs a so-called *ε-box dominance archive* fitness assignment that improves the convergence and diversity throughout the search as defined below.

For a given $\varepsilon > 0$, a vector (objective values) $\boldsymbol{u} = (u_1, u_2, \dots, u_m)$ ε-box dominates another vector $\boldsymbol{v} = (v_1, v_2, \dots, v_m)$ if and only if one of the following occurs:

1- $\left\lfloor \frac{\boldsymbol{u}}{\varepsilon} \right\rfloor \prec \left\lfloor \frac{\boldsymbol{v}}{\varepsilon} \right\rfloor, Or$

2- $\left\lfloor \frac{\boldsymbol{u}}{\varepsilon} \right\rfloor = \left\lfloor \frac{\boldsymbol{v}}{\varepsilon} \right\rfloor \ and \ \left\| \boldsymbol{u} - \varepsilon \left\lfloor \frac{\boldsymbol{u}}{\varepsilon} \right\rfloor \right\| < \left\| \boldsymbol{v} - \varepsilon \left\lfloor \frac{\boldsymbol{v}}{\varepsilon} \right\rfloor \right\|$

Borg MOEA executes an update archive procedure for every generated solution to add the solutions that ε-box dominate all solutions in the archive to the archive. Abstractly, the ε -box dominance archive provides a minimum search resolution by dividing the objective space into hyper-boxes with side length ε. This is useful when decision-makers can define their precision goals (e.g., 1000$ for construction costs) or computational limits.

Moreover, Borg includes different design principles and several novel components that improve its efficiency for handling many-objective, extremely multimodal challenging real-world problems. The main components are as follow:

1. **ε- Progress** that evaluates the search progress and prevent stagnation in search. ε-Progress happens when a solution d is accepted into the archive such that no existing member of the archive existed with the same ε -box index vector.
2. **Restart mechanisms** that revive search after a stagnation is detected using ε-progress. These mechanisms include (1) using adaptive population size, (2) using adaptive tournament selection size and (3) emptying population and repopulating using solutions from the archive.
3. **Auto-Adaptive Multi-Operator Recombination** that enhances search in a wide assortment of problem domains. This feature systematically recognizes operators that produce more thriving offspring and rewards them by increasing the number of offspring generated by that operator. Borg includes six different recombination operators: *Simulated Binary Crossover*, *Differential Evolution*, *Parent-Centric Crossover*, *Simplex Crossover*, *Unimodal Normal Distribution Crossover*, and *Uniform Mutation*.

The Borg MOEA combines the above-mentioned components in its main exploration algorithm as presented in the following.

1. An initial population is randomly generated within the feasible search space.
2. For all chromosomes, the corresponding objective functions are evaluated. The ε -box dominance condition is checked for all generated solutions and the archive is updated.
3. Using the auto-adaptive multi-operator recombination procedure, one of the recombination operators is selected.
4. For a recombination operator requiring k parents, one parent is selected uniformly at random from the archive. The remaining $k-1$ parents are selected from the population using tournament selection.
5. The generated offspring solutions are evaluated and considered for inclusion in the population and archive.
6. After a user-defined number of iterations, ε-Progress and the population-to-archive ratio are checked. If a restart is needed, the main loop pauses and the restart procedure is executed. Once the restart has completed, the algorithm with a new population is repeated from step 2. This process repeats until termination.

TOPSIS

Multi-criteria decision analysis (MCDA) techniques are utilized to choose the most desirable alternative from a finite assortment of decision alternatives in terms of multiple, often conflicting criteria. There are many MCDA methods, such as MAXMIN, MAXMAX, SAW, AHP, TOPSIS, SMART and ELECTRE. The choice of methods can be determined based on diverse criteria such as the nature of the decision-making problem being addressed (ranking, sorting or choosing, ease of use, data requirements, computation time and human resource requirements and software availability [77].

In this thesis, TOPSIS (Technique for Order of Preference by Similarity to Ideal Solution) is selected for the aid of decision-making and ranking the Pareto-optimal solutions found by MOO in Chapter 5. TOPSIS was firstly proposed by [78] and later developed by [79] and [80]. TOPSIS ranks the alternatives based on the relative similarity to the ideal solution, which avoids the circumstances of having the identical similarity index to both positive ideal and negative ideal solutions [77]. The TOPSIS is a practical technique with an intuitive and clear logic that represents the rationale of human choice. It can be straightforwardly applied and enjoys high computational efficiency [81]. The main steps of TOPSIS include [32]:

1. Computing a normalized decision matrix;
2. Computing the weighted normalized decision matrix;
3. Recognizing so-called positive-ideal and negative-ideal alternatives;
4. Computing separation (distance) measures, using the n-dimensional Euclidean distance;
5. Computing the relative closeness to the positive-ideal solution;
6. Rank preference order based on their TOPSIS score (the higher TOPSIS score, the better alternative).

As the determination of a specific weight for each index in TOPSIS is usually subjective, the entropy method was utilized to calculate the weights and reduce the subjectivity [82]. Entropy is a term in information theory introduced by [83]. The calculation steps of the entropy method for weight determination are as follows, supposing a decision matrix x_{ij} with m alternatives (rows) and n indicators (columns) [32]:

1. Calculate the normalized p_{ij} of the i^{th} alternatives to the j^{th} indicator in decision matrix:

$$p_{ij} = {x_{ij}}/{\sum_{i=1}^{m} x_{ij}} \qquad (1 \leq i \leq m,\ 1 \leq j \leq n) \tag{2.17}$$

2. Calculate the output entropy e_j of the j^{th} factor:

$$e_j = -k \sum_{i=1}^{m} p_{ij} \ln(p_{ij}) \qquad (k = {1}/{\ln(m)},\ 1 \leq j \leq n) \tag{2.18}$$

3. Calculate the weight of entropy w_j:

$$w_j = {(1 - e_j)}/{\sum_{j=1}^{n}(1 - e_j)} \qquad (1 \leq j \leq n) \tag{2.19}$$

Chapter 3. Hanging gardens algorithm for the optimization of decentralized UDSs

Most of the content of this chapter has been published in the *Journal of Water Resources Planning and Management* under the title “Hanging Gardens Algorithm to Generate Decentralized Layouts for the Optimization of Urban Drainage Systems” [41], and as a book chapter in Mannina G. (eds) New Trends in Urban Drainage Modelling under the title “A Graph-Theory Based Algorithm to Generate Decentralized Urban Drainage Layouts” [40].

Summary

Traditional urban drainage systems rely heavily on centralized network-based infrastructures. Recently, the idea of centralized urban drainage networks has increasingly been questioned. The latest investigations suggest a transition from centralized to decentralized or hybrid schemes. Therefore, there is a need for tools and methodologies to evaluate and optimize drainage networks with arbitrary DC. For this purpose, I have developed an algorithm called the *hanging gardens algorithm* to generate all possible sewer layouts and to explore different degrees of decentralization. The proposed algorithm starts with generating a centralized layout and introducing a list of outlet candidates. Next, it adds arbitrary outlets from candidates to the generated layout and uses a graph-theory based approach to assign parts of the layout to different outlets. This procedure is iterated until all (combinations) outlet candidates have been included. To demonstrate the performance of the proposed algorithm in enumerating all different DC and generating realistic layouts, the algorithm is coupled with an optimization engine in order to optimize the stormwater collection network of a section of the city of Ahvaz in Iran. The number and location of outlets, the layout configuration of each part and the size of pipes are used as optimization variables to minimize costs subject to hydraulic and feasibility constraints. The proposed algorithm performs well in exploring different DC and finding near-optimum solutions.

3.1 Introduction

As mentioned in Chapter 1, recent investigations suggest a transition from centralized urban water management to decentralized or hybrid schemes [7, 8, 15, 16]. By the definition adopted from Eggimann et al. (2015) [7], the DC of a system is being increased as the number of elements that are linked to it and interconnected are being increased.

Still, a number of restrictions make fully decentralized systems with space-consuming on-site treatment ideas almost impossible, especially in dense urban centers. Therefore, there is a need for robust methodologies to assess and optimize the performance of all systems: decentralized, hybrid and centralized [8, 9, 84]. For this purpose, I developed a new algorithm, the *hanging gardens algorithm* in this chapter to generate all feasible urban drainage schemes with an arbitrary DC systematically.

The *hanging gardens algorithm* is novel and tailor-made for satisfying all constraints of designing an urban drainage pipe network. This algorithm can be equipped with appropriate optimization algorithms to find the optimum configuration in conjunction with the optimum DC. The focus of this chapter is on decentralized layout generation because this is the basis of any network-based infrastructure. Considering other alternatives (including green infrastructures) is done in Chapter 4. The proposed algorithm starts with generating a centralized layout. In order to generate an initial centralized layout with an arbitrarily selected outlet, the *loop-by-loop cutting algorithm* with some modifications proposed by Haghighi (2013) [53] is adopted. Then, the *hanging gardens algorithm* adds other outlets from a list of candidate outlets to the generated graph and divides it into decentralized parts. Through further iteration controlled by a coupled optimization algorithm, it can explore layouts with a varying number of outlets while accounting for all combinations of candidate outlets.

A real case study, the stormwater collection network of a part of the city of Ahvaz in Iran, is designed in this chapter using the proposed approach. For this, an optimization problem is formulated and the *hanging gardens algorithm* is coupled with an optimization engine. The number and location of the outlets, the layout configuration of each part and the size of the pipes are considered as optimization variables. A single objective optimization for the cost is done and obtained results are discussed.

The structure of this chapter is defined as follows: the next section gives the mathematical formulation of the problem and the state of the literature on generating different kinds of urban drainage layouts. The section after that presents algorithms for generating layouts with an arbitrary DC. A real case study is then presented to assess the performance of our approach. The final section concludes this study by discussing the advantages and disadvantages of this work and by suggesting possible topics for further research.

3.2 Problem review and formulation

Designing a sewer network consists of two sub-problems; (1) generating the layout configuration of the sewer network and (2) designing the sewers hydraulically [52, 85, 86]. As problems involving optimization, these sub-problems are nonlinear and discrete in nature and include many complex hydraulic and technical constraints [52]. In general, the mathematical least-cost optimization of sewer networks could be formulated as:

$$\boldsymbol{d}_{\text{opt}} = \arg \min_{\boldsymbol{d} \in \boldsymbol{D}} [f_{\text{cost}}] \tag{3.1}$$

where $\boldsymbol{d}_{\text{opt}}$ is the optimal choice for the decision variable $\boldsymbol{d}$ that defines the sewer system, including layout configuration and hydraulic specifications as shown in equation 2. $\boldsymbol{D}$ is the feasible space where both layout and hydraulic constraints are satisfied, and f_{cost} is the cost function. The multi-objective versions of equation 1 will be pursued in Chapter 5.

There are, in general, three different approaches in the literature to solve the aforementioned problem [87]: (1) to optimize the layout irrespective of the later hydraulic design (2) to solve the layout and hydraulic design simultaneously and (3) to optimize the hydraulic design for a fixed layout. The various optimization models used for these purposes are discussed in detail in [52, 86, 87]. In all of these studies, the number and location of outlets are pre-defined.

The main aim of this chapter is to include DC into the optimization problem. Therefore, $\boldsymbol{d}$ in equation 1 is defined as follow:

$$\boldsymbol{d} = \begin{bmatrix} DC, layout\ parameters, pipe\ diamaters, \\ pipe\ slopes, pump\ stations \end{bmatrix} \tag{3.2}$$

While layout parameters express the connectivity between different components (sewers and pumping stations) connected to one outlet each, the DC implicitly explains how the system as a whole is distributed. Still, there is no clear definition of how to measure DC in the field of urban water management [7]. Eggimann et al. (2015) [7] adopted a weighted DC by taking into account a continuum of possible facility sizes for wastewater management infrastructures. As the focus of this chapter is on generating decentralized layouts with a varying number of outlets, DC is simply defined as follows:

$$\text{DC} = 100 \times \left(1 - \frac{N_{SO} - 1}{N_{PO} - 1}\right) \quad (\%) \tag{3.3}$$

where N_{SO} is the number of selected outlets from a list of candidates, and N_{PO} is the number of possible candidate outlets. Using this definition, the DC is zero when all outlet candidates are selected (totally decentralized) and is one when only one outlet is selected (totally centralized). In the following paragraphs, the literature on generating different kinds of urban drainage layouts is recapitulated to indicate the gap I will address and to clarify the main contribution of this chapter.

Mathematically, a feasible sewer network layout is an arborescence-spanning tree with a root that is finally connected to a wastewater treatment plant, to a primary sewer collection line, or a water body. In cases with more than one outlet, a forest of arborescences represents the layout [53]. To do a sewer network optimization systematically, a layout generator algorithm is required to satisfy all constraints while extracting layout alternatives from an initially fully-connected base graph. The base graph contains all possible sewer lines and is undirected, i.e. it is fully connected, cyclic in parts and does not yet set the flow directions.

Concerning generating centralized sewer layouts, a limited number of methods can be found in the literature. For example, Tekel and Belkaya (1986) [88] applied the so-called *shortest-path spanning tree method* (from the mathematical field of graph theory) to generate different layouts during the optimization. Li and Matthew (1990) [89] introduced a so-called *searching direction method* that was able to create the shortest spanning tree of a looped graph to generate the layouts. Moeini and Afshar (2012) [90] employed a *tree growing algorithm* to simultaneously solve the layout and sewer design sub-problems. Haghighi (2013) [53] introduced an adaptive layout generator called the *loop-by-loop cutting algorithm*. Using this algorithm, the base graph is opened with a systematic procedure while the layout constraints are systematically met. This converts the base graph to a tree that is a feasible layout.

The following brief overview of methods that are capable of generating decentralized layouts demonstrates that few algorithms have thus far been developed:

Madkour et al. (2017) [91] introduced a graph-theory based algorithm that gives a near-optimal solution to a graph-partitioning problem, describing the problem as follows: "Given an edge-weighted, undirected graph G and a positive integer k, the desire is to find k subtrees T_1, . . . , T_k within G such that (1) each vertex of G is contained in T_i for some i and (2) the maximum weight of any T_i is as small as possible."

This algorithm exhibits good efficiency in finding a near-optimal solution for the described problem; however, it is not possible to employ it in the field of urban water management because of some limitations such as using fixed weight for the edges (pipes).

Diogo et al. (2000) [92] considered hybrid solutions and used simulated annealing to minimize cost for the regional wastewater system planning. To generate hybrid sewer configurations, the so-called *forest algorithm* based on the *growing spanning tree algorithm* was adopted. The layout and sewer design sub-problems were then solved separately. Eggimann et al. (2015) [7] developed a planning tool for sustainable network infrastructure planning (SNIP) to find the optimal DC for wastewater infrastructures. The optimal number, placement and sizing of wastewater treatment facilities as well as the layout of gravity-driven and pressurized sewer networks were considered as optimization variables. The method from Eggimann et al. (2015) [7] used the *shortest path-finding* and *clustering algorithms*. This is a very promising way to find the optimal DC especially for local-scale problems; however, some shortcomings restrict its performance for more detailed engineering projects and regional-scale problems. For example, the *greedy algorithms* used in SNIP do not guarantee to find an optimal global solution, particularly in flat areas where the optimum layout is independent of ground topography. Furthermore, it is not possible to consider the existing network infrastructures in SNIP.

Jung et al. (2018) [93] compared the cost of centralized and decentralized wastewater management in an urban town in India. To generate different potential decentralized system configuration, a simplified sewer model (instead of the hydraulic simulation) and mixed-integer programming optimization (to minimize the sum of sewer distances between the wastewater sources and their respective WWTPs) were used. However, these simplifications are far from real applications.

To mitigate the above-mentioned shortcomings especially in the area of layout-generating algorithms, the current study introduces the so-called *hanging gardens algorithm*. It is based on graph theory to generate urban drainage schemes that represent real sewer networks with an arbitrary DC. The main advantages of the proposed framework are:

- The optimum DC can be explicitly considered as an optimization variable in the mathematical optimization formulation.

- As the mathematical representation of generated networks is close to real sewer systems, the proposed algorithm can be used for any required level of detail and regional or local infrastructure planning.
- The *hanging gardens algorithm* can automatically handle all layout constraints. Therefore, it can be coupled with any optimization algorithm or external hydraulic simulation code and software.
- The layout and hydraulic design sub-problems can be solved simultaneously.
- The existing infrastructures and other possible decentralization measures (e.g. green infrastructures) can be considered within the proposed framework.

3.3 Layout generator for centralized and decentralized systems

The proposed approach involves two algorithms for generating a decentralized layout, the *modified loop-by-loop cutting algorithm* and the *hanging gardens algorithm* (developed in this chapter). The *loop-by-loop cutting algorithm* generates a centralized layout, and the *hanging gardens algorithm* uses this generated layout to produce a decentralized layout. The procedure is shown schematically in Figure 3.1.

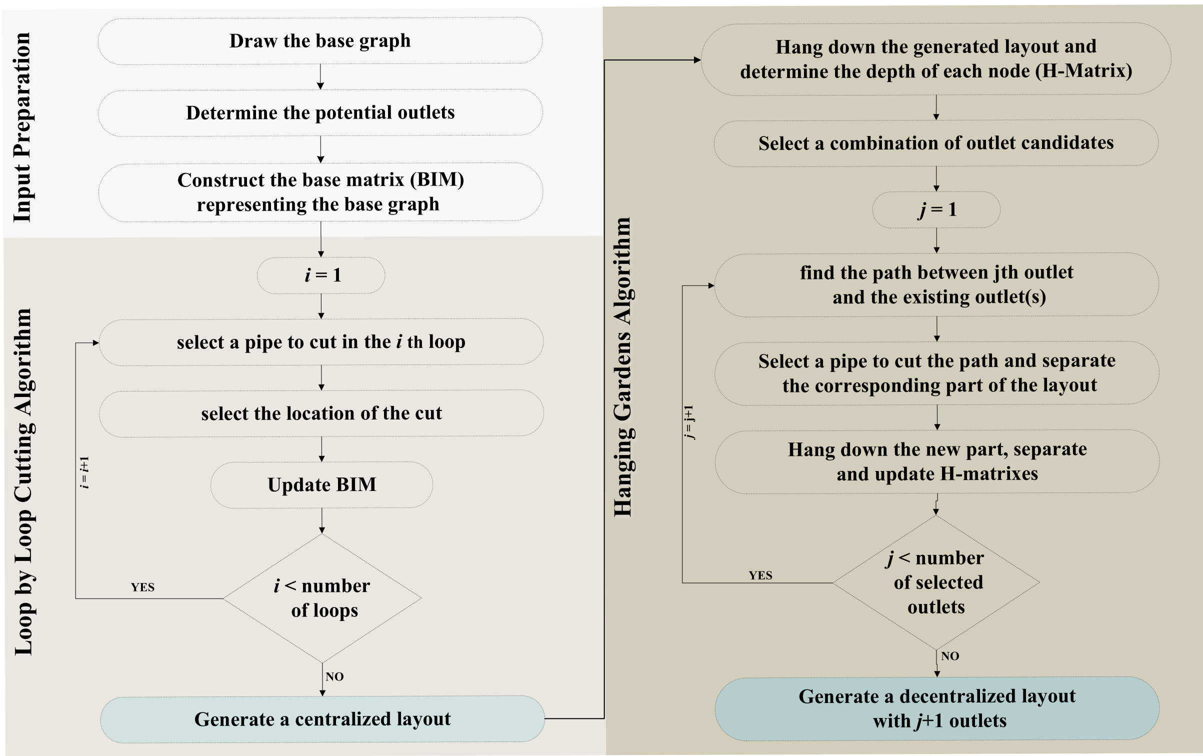


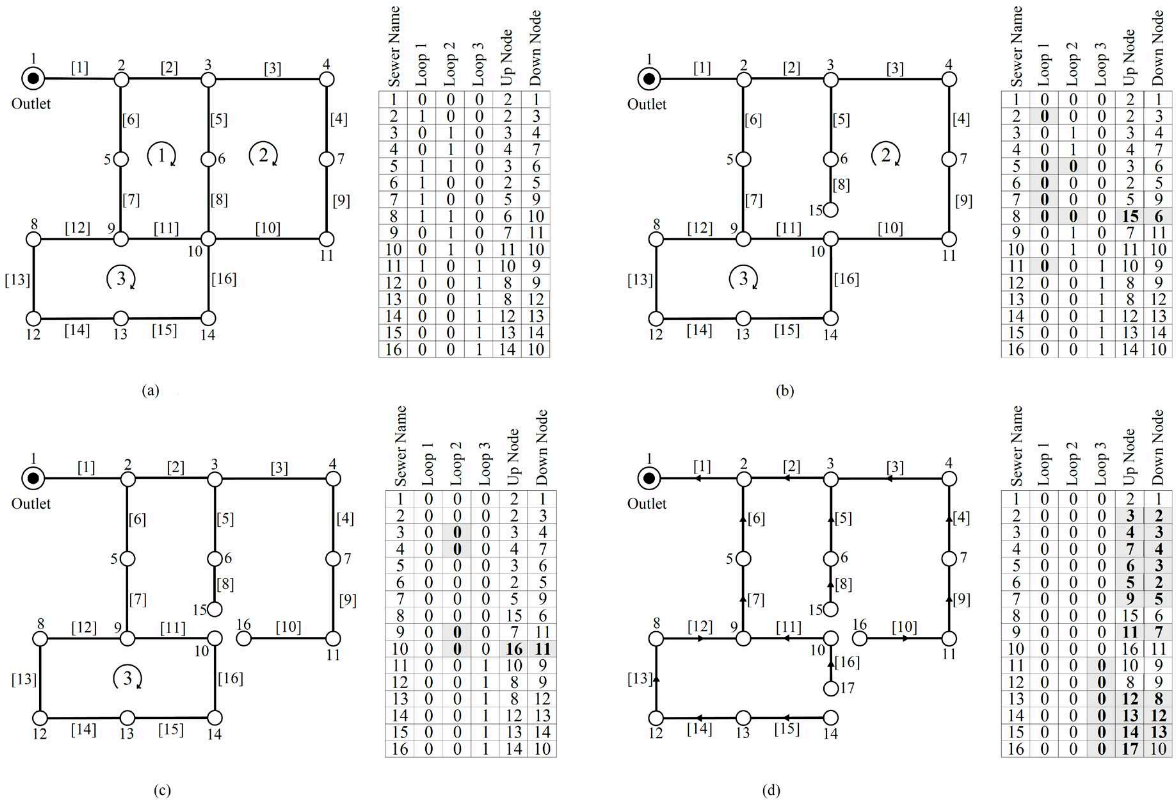
Figure 3.1: *The proposed framework to generate decentralized layouts*

3.3.1 *Loop-by-loop cutting algorithm* for centralized layout

The *hanging gardens algorithm* introduced in the next section will use the centralized layout generated by the *loop-by-loop cutting algorithm* as its input. The remainder of this section describes the *loop-by-loop cutting algorithm* with some modifications in short and is mainly adopted from [54].

The first step is to provide a base graph for the sewer system at hand. This base graph includes all drainage possibilities concerning the street alignments, topology, barriers, watercourses, locations of the outlets and existing sewers in the city, as shown in Figure 3.2(a). Sewers and manholes are then named with integer numbers as unique identifiers as seen in Figure 3.2(a). A matrix, in this case, called the **B-matrix**, is used to encode the base graph mathematically. The **B-matrix** consists of m rows and $NL + 3$ columns, where m and NL are the number of pipes and loops in the base graph, respectively (Figure 3.2-a). Inside the **B-matrix**:

- Column 1 contains the sewer names,
- Columns 2 to $NL + 1$ contain the sewer-in-loop indicators which indicate whether a sewer is in a loop (value 1) or not (value 0).
- Columns $NL + 2$ to $NL + 3$ contain the names of the sewer ends (manhole names).



(a)

Sewer Name	Loop 1	Loop 2	Loop 3	Up Node	Down Node
1	0	0	0	2	1
2	1	0	0	2	3
3	0	1	0	3	4
4	0	1	0	4	7
5	1	1	0	3	6
6	1	0	0	2	5
7	1	0	0	5	9
8	1	1	0	6	10
9	0	1	0	7	11
10	0	1	0	11	10
11	1	0	1	10	9
12	0	0	1	8	9
13	0	0	1	8	12
14	0	0	1	12	13
15	0	0	1	13	14
16	0	0	1	14	10

(b)

Sewer Name	Loop 1	Loop 2	Loop 3	Up Node	Down Node
1	0	0	0	2	1
2	**0**	0	0	2	3
3	0	1	0	3	4
4	0	1	0	4	7
5	**0**	**0**	0	3	6
6	**0**	0	0	2	5
7	**0**	0	0	5	9
8	**0**	**0**	0	**15**	**6**
9	0	1	0	7	11
10	0	1	0	11	10
11	**0**	0	1	10	9
12	0	0	1	8	9
13	0	0	1	8	12
14	0	0	1	12	13
15	0	0	1	13	14
16	0	0	1	14	10

(c)

Sewer Name	Loop 1	Loop 2	Loop 3	Up Node	Down Node
1	0	0	0	2	1
2	0	0	0	2	3
3	0	**0**	0	3	4
4	0	**0**	0	4	7
5	0	0	0	3	6
6	0	0	0	2	5
7	0	0	0	5	9
8	0	0	0	15	6
9	0	**0**	0	7	11
10	0	**0**	0	**16**	**11**
11	0	0	1	10	9
12	0	0	1	8	9
13	0	0	1	8	12
14	0	0	1	12	13
15	0	0	1	13	14
16	0	0	1	14	10

(d)

Sewer Name	Loop 1	Loop 2	Loop 3	Up Node	Down Node
1	0	0	0	2	1
2	0	0	0	**3**	**2**
3	0	0	0	**4**	**3**
4	0	0	0	**7**	**4**
5	0	0	0	**6**	**3**
6	0	0	0	**5**	**2**
7	0	0	0	**9**	**5**
8	0	0	0	15	6
9	0	0	0	**11**	**7**
10	0	0	0	16	11
11	0	0	**0**	10	9
12	0	0	**0**	8	9
13	0	0	**0**	**12**	**8**
14	0	0	**0**	**13**	**12**
15	0	0	**0**	**14**	**13**
16	0	0	**0**	**17**	10

Figure 3.2: *Example base graph and cutting procedure*

One sewer of each loop in the base graph must be cut to create a tree-like layout. When a pipe was selected for cutting, it can be cut either at its upstream or at its downstream manhole. As a result, there are two decision variables for opening each loop including the cut-pipe name and the cut location. These two decision variables are called α and β in the algorithm. In practical applications, these variables are handled by an optimization engine. For each loop i, α_i is a real number on the interval $(0, 1)$, and β_i is a binary number that either assumes 0 or 1 as values. The value of β decides whether the pipe is cut from its upstream or downstream end.

While β values directly indicate how a pipe is cut, the α values must be decoded first to find the cut-pipes. As seen in Figure 3.2(a), there is a column for every loop (columns 2 to NL+1). The non-zero members in each loop's column point to the name of possible sewers to be cut. By the following equation, each α_i is mapped to an integer number μ_i that enumerates the cut-pipe in loop i.

$$\mu_i = round\left(1 + \alpha_i\left(\sum_{j=1}^{m} B_{j,i+1} - 1\right)\right) \tag{3.4}$$

Here, $\sum_{j=1}^{m} B_{j,i+1}$ is the total number of pipes in loop i that can be cut; the function "round" returns the closest integer value to the parenthesis, and $B_{j,i+1}$ are the elements of the base graph matrix. For example, let us consider the base graph of Figure 3.2(a) and its **B-matrix**. For opening loop 1, the number of all possible cut-pipes is 6 (the summation of all non-zero members in the column of loop 1). If, for example, $\alpha_1 = 0.82$ then $\mu_1 = round\big(1 + 0.82(6-1)\big) = 5$ which means that the fifth sewer (sewer 8) with a non-zero value in the column of loop 1 (column 2 in this case) is selected to be cut. Then, the selected sewer is cut at one of its ends, manholes 5 or 9, depending on the β value. After a loop is opened, the base graph changes and the **B-matrix** must be updated accordingly. Subject to the constraints of the layout of the sewer, the **B-matrix** must be modified at each step (after each loop opening) for:

• ***The new manhole.*** As a sewer line is cut for loop i, a new manhole will appear at the truncation end. The new manhole is named with the number $n + i$ and is substituted in the **B-matrix** for the manhole located at the previous end; n is the number of main manholes in the base graph. For example, in Figure 3.2(b) manhole 15 is added to the upstream end of sewer 8, and the value of the element $B_{8,5}$ is changed from 10 to 15.

• ***The flow direction.*** In the cut-pipe, the sewage flows from its new manhole $n + i$ toward the other end. This means that the new manhole must be located at the upstream end of the pipe in the B-matrix. For this purpose, the situation of the new manhole $n + i$ in the base graph is updated in column $NL + 2$ if it is not already there.

• ***Once pipe cutting.*** If a sewer is cut to open a loop, it is no longer possible to be chosen for the other loops. This constraint is met by switching all the nonzero sewer-in-loop indicators to zero in the row of the newly cut-pipe. For example, in the B-matrix of Figure 3.2(b), after pipe 8 was cut for loop 1 in Figure 3.2(b), all members of $B_{8,2\ to\ 4}$ are changed to zero.

• ***Network integrity.*** In case all sewers linked to a manhole can be cut for different loops (like sewers 8, 10, 11 and 16 in Figure 3.2(a) that are all attached to manhole 10), at least one link must remain to drain the common manhole and to keep the network integrity. To satisfy this limitation, some actions in each step must be taken:

- The downstream manhole of the cut pipe (e.g. node 6 in Figure 3.2-b) is checked. If there is exactly one remaining pipe (pipe 5 in Figure 3.2-b) connected to this manhole, the remaining pipe must not be cut in the next loops. Thus, all elements of B in the row that represents the remaining pipe from columns 2 to $NL + 1$ are set to zero ($B_{5,2\ to\ 4}$ in Figure 3.2-b)
- The downstream manhole of the remaining pipe as described in I (e.g. pipe 9 and manhole 7 in Figure 3.2-c) is also checked for the same condition. If there is exactly

one pipe connected to this manhole, it must not be cut in the next loops. This procedure is iterated until all problematic pipes are detected (e.g. pipes 4 and 3 in Figure 3.2-c).

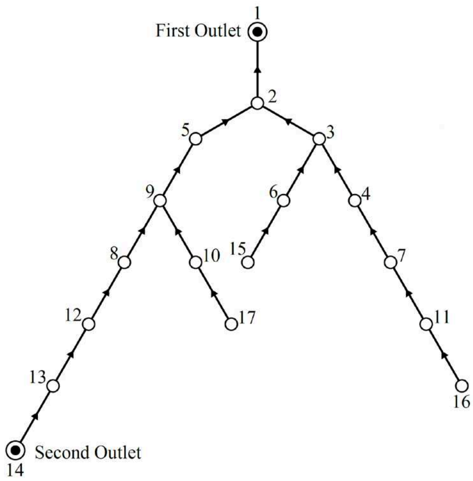
- The corresponding node to the new node in the cut link is also checked for this limitation (e.g. node 14 in Figure 3.2-d). If there is exactly one pipe connected to this manhole, it must not be cut in the next loops. Similar to the last step, this step is iterated until all problematical pipes in this direction are detected (e.g. pipes 15, 14 and 13 in Figure 3.2-d).

This update permits the model to split a street not only at the edges but also at any other section. This is important when the model represents the sewer system in a more detailed way than at street level, i.e., if the model includes all real inlets and manholes.

After the above updates are done, the next loop is targeted and the procedure is continued until all NL loops are opened. Ultimately, a feasible sewer layout with m sewers, $n + NL$ manholes (where $m = n + NL - 1$) and with no loop is generated based on the defined α and β variables. Afterward, the sewer directions are set in the direction of the outlet manhole based on the principle that "except for the outlet exactly one sewer leaves every manhole". Each sewer direction is modified in the B-matrix in such a way that the upstream manholes are allocated in column $NL + 1$ and the downstream ones are allocated in column $NL + 2$ (Figure 3.2-d). It is worthwhile to mention that any arbitrary set of inputs and location of the outlet always results in a feasible directed tree using the described method.

3.3.2 *Hanging gardens algorithm* for an arbitrary DC

In the previous section, the procedure of generating a feasible layout with one outlet (centralized layout) from a base graph was described in brief. Now, the question is how one can divide a directed tree into several ones when new possible outlet locations are introduced to the current layout. In this section, the proposed approach is described using an example. Suppose node 14 in Figure 3.2(d) has the potential to be treated as an outlet. It could be proved that, when selecting any node of a tree as a candidate for a new root to a tree, there will be one and just one path between the both of the existing root and the new root. Therefore, the problem is a matter of finding the path and cutting it to generate two separate trees. Since the output of the *loop-by-loop cutting algorithm* is a directed tree, one can easily find the path by following the direction of the pipes from the new outlet (node 14) to the first outlet (node 1 in Figure 3.2-d). However, in this simple approach, it is almost impossible to systematically find out, with only the path and B-matrix, which nodes and pipes are contributing to which part after separation (see Figure 3.2-d). The problem gets worse when the network is more complex. To handle this problem, an innovative approach is proposed in this chapter. Suppose the generated centralized layout (Figure 3.2-d) is a system of balls and ropes with mass (the nodes in the spanning tree are the balls and the edges are the ropes with similar length). If this system was hung down from its root, the result would be a hanging tree as shown in Figure 3.2(d). By doing this, it is possible to sort the nodes of the layout in a hierarchically way. For this purpose, a number is assigned to each node that indicates its depth i.e. the number of edges to its root (node 1). The nodes are sorted according to this number from the lowest level to the highest level in another matrix, also known as the **H-matrix**. The **H-matrix** consists of n rows and $n + 2$ columns where n is the number of the nodes in the generated layout. Inside the **H-matrix:**



Level	Node Name	Node Name: 1	2	3	5	4	6	9	7	8	10	15	11	12	17	13	16	14
0	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1	2	1	0	1	1	0	0	0	0	0	0	0	0	0	0	0	0	0
2	3	0	1	0	0	1	1	0	0	0	0	0	0	0	0	0	0	0
2	5	0	1	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0
3	4	0	0	1	0	0	0	0	1	0	0	0	0	0	0	0	0	0
3	6	0	0	1	0	0	0	0	0	0	0	1	0	0	0	0	0	0
3	9	0	0	0	1	0	0	0	0	1	1	0	0	0	0	0	0	0
4	7	0	0	0	0	1	0	0	0	0	0	0	1	0	0	0	0	0
4	8	0	0	0	0	0	0	1	0	0	0	0	0	1	0	0	0	0
4	10	0	0	0	0	0	0	1	0	0	0	0	0	0	1	0	0	0
4	15	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0
5	11	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	1	0
5	12	0	0	0	0	0	0	0	0	1	0	0	0	0	0	1	0	0
5	17	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0
6	13	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	1
6	16	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0
7	14	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0

Figure 3.3: *H-matrix of a hanging tree*

- Column 1 contains the node levels,
- Column 2 contains the node names,
- Columns 3 to $n + 2$ contain the node connectivity information, which indicates whether a node is connected to another node (value 1) or not (value 0).

The **H-matrix** for the current example is presented in Figure 3.3. Notice that the **H-matrix** is based on the nodes while the **B-Matrix** was based on the pipes.

The *hanging gardens algorithm* was named accordingly because it cuts the initial hanging tree into many, smaller hanging trees. It generates decentralized layouts employing three different modules:

- A module to find a path between the proposed additional new root and the existing root(s) (pathfinder)
- A module to select the location for cutting that path and to separate the graph into two parts (separator)
- A module to find contributing nodes and pipes in each part and construct an **H-matrix** for each of the new trees (matrix constructor)

Figure 3.3 and Figure 3.4 show this procedure. All three modules are described below using an example.

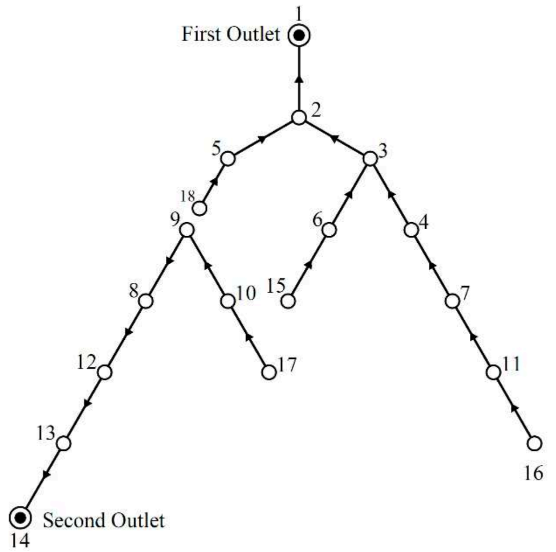
- ***Pathfinder.*** To find the path between the suggested new root and the old root, it is enough to follow the direction of pipes from upstream to downstream. In the **H-matrix**, the first non-zero element before the main diagonal of each row presents the downstream node of the corresponding node. Therefore, the algorithm starts from the new root (node number 14 in Figure 3.3), finds the downstream node and leaps to the corresponding row. This procedure is iterated until the algorithm reaches the old root.
- ***Separator.*** The output of the Pathfinder module is a sequence of nodes from the new root to the old root as shown below.
  $\text{path} = [13,12,8,9,5,2]$
  The separator module employs equation 6 to select a node for a cut in the path:

$$\mu = round\left(1 + \gamma \times \left(N_{path} - 1\right)\right) \tag{3.5}$$

Here, γ is a real number on the interval (0,1) like α in equation 5, and N_{path} is the number of nodes in the path. For example, if $\gamma = 0.6$ and $N_{path} = 6$, then $\mu = 4$, which means the forth node in the path, node number 9, is selected to separate the tree in Figure 3.4(a). The values of γ for each separation are again handled by an optimization engine.

- ***Matrix constructor.*** After separating the layout, the **H-matrix** must also be separated and updated to represent the new layout. The largest task is to recognize which elements belong to which part. For this purpose, the matrix constructor module starts searching for the nodes in the new part. The other nodes must remain in the old part(s). Here, the first node in the new part is the upstream node of the cut pipe (node 9 in Figure 3.4-a). Because this node has the lowest level among all nodes contributing to the new part, going against the old flow direction identifies all the nodes in the new part. Each non-zero element after the main diagonal of the initial **H-matrix** represents upstream nodes connected to the node at hand. Therefore, this module starts from the corresponding row of the first node in the new part (node 9 and row 7 in Figure 3.3); recognizes upstream nodes of it (nodes 8 and 10), goes to the corresponding rows of recently recognized nodes (rows 9 and 10 in Figure 3.3) and checks whether there are any further nodes upstream. After recognizing all nodes in the new part, two **H-matrixes** must be constructed, one for each part. To construct the **H-matrix** for the old part, it is enough to eliminate all rows and columns corresponding to the nodes that have been moved to the new part. For the new part, all nodes must be redirected into the new manhole. After that, the obtained directed layout must be sorted again according to the distance between nodes and the root, as explained before, by hanging down the new tree from its root (Figure 3.4-a).

Using the proposed algorithm, it is possible to separate any directed tree into two directed trees. By introducing a new root to the system (e.g. node 16 in Figure 3.4-b), firstly, the algorithm searches to find out in which part the new root exists and then uses the described modules to divide it. A vector of binary variables, ζ, is used here to indicate which outlets from the candidate outlets are included in the generated layout. This procedure can be iterated until all the possible roots (and all possible combinations) are included in the final layout. As a final step, all the changes are applied to the real layout in the base graph. Figure 3.5 depicts a decentralized layout generated by the proposed algorithm.

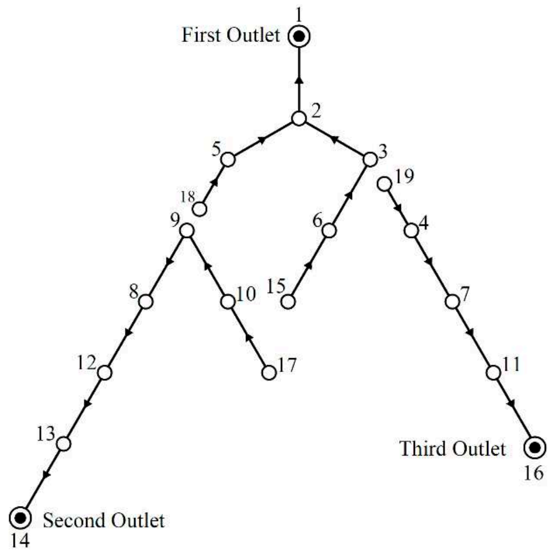


Level	Node Name	1	2	3	5	4	6	18	7	15	11	16
0	1	0	1	0	0	0	0	0	0	0	0	0
1	2	1	0	1	1	0	0	0	0	0	0	0
2	3	0	1	0	0	1	1	0	0	0	0	0
2	5	0	1	0	0	0	0	1	0	0	0	0
3	4	0	0	1	0	0	0	0	1	0	0	0
3	6	0	0	1	0	0	0	0	0	1	0	0
3	18	0	0	0	1	0	0	0	0	0	0	0
4	7	0	0	0	0	1	0	0	0	0	1	0
4	15	0	0	0	0	0	1	0	0	0	0	0
5	11	0	0	0	0	0	0	0	1	0	0	1
6	16	0	0	0	0	0	0	0	0	0	1	0

(Node Name)

		14	13	12	8	9	10	17
0	14	0	1	0	0	0	0	0
1	13	1	0	1	0	0	0	0
2	12	0	1	0	1	0	0	0
3	8	0	0	1	0	1	0	0
4	9	0	0	0	1	0	1	0
5	10	0	0	0	0	1	0	1
6	17	0	0	0	0	0	1	0

(a)



(Node Name)

Level	Node Name	1	2	3	5	6	18	15
0	1	0	1	0	0	0	0	0
1	2	1	0	1	1	0	0	0
2	3	0	1	0	0	1	0	0
2	5	0	1	0	0	0	1	0
3	6	0	0	1	0	0	0	1
3	18	0	0	0	1	0	0	0
4	15	0	0	0	0	1	0	0

		14	13	12	8	9	10	17
0	14	0	1	0	0	0	0	0
1	13	1	0	1	0	0	0	0
2	12	0	1	0	1	0	0	0
3	8	0	0	1	0	1	0	0
4	9	0	0	0	1	0	1	0
5	10	0	0	0	0	1	0	1
6	17	0	0	0	0	0	1	0

		16	11	7	4	19
0	16	0	1	0	0	0
1	11	1	0	1	0	0
2	7	0	1	0	1	0
3	4	0	0	1	0	1
4	19	0	0	0	1	0

(b)

Figure 3.4: *Decentralizing procedure using the proposed algorithm*

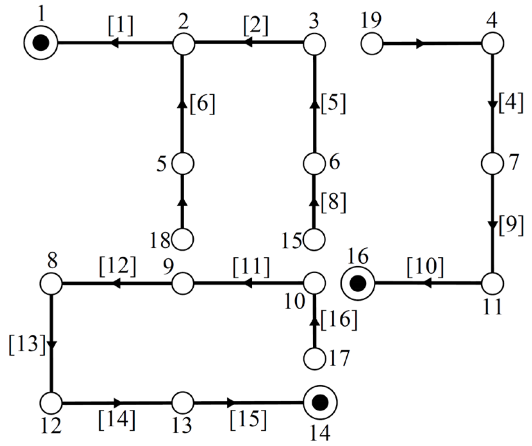


Figure 3.5: *Example Decentralized layout generated by the hanging gardens algorithm*

Because there is no limitation to the number of candidate outlets (all nodes can be introduced as a possible outlet in the model), the *hanging gardens algorithm* is capable of generating all kinds of layouts from fully centralized to fully decentralized. In case of steep terrain, where the tree-like layout can be determined using engineering judgment, the first step (generating a centralized layout) can be skipped. In place of this first step, the *hanging gardens algorithm* simply uses a fixed layout to generate different scenarios (number and location of possible outlets).

3.4 Application of the proposed algorithm

3.4.1 Case study

To demonstrate the performance of the proposed algorithm in enumerating all different DC and generating realistic layouts, the proposed algorithm is coupled to an optimization engine and applied to design the stormwater network of a part of the city of Ahvaz in Iran (Figure 3.6). Ahvaz is located in the southwest of Iran with a population of more than one million. The city has a semi-desert climate with long and very hot summers and short and mild winters. Every year, urban flooding due to a lack of a proper stormwater management system results in several problems, such as public inconvenience, economic and environmental destruction and the spread of infectious diseases. Recent efforts to design a centralized stormwater network for the city have not had any success because of the following reasons: The city is totally flat, the design flow is relatively high, the groundwater level is high and salty (large-diameter sewers are not possible), and the city could not afford the high initial investment for a centralized system. Therefore, piecewise decentralized development is the only promising approach.

Figure 3.6: *The case study (Google maps)*

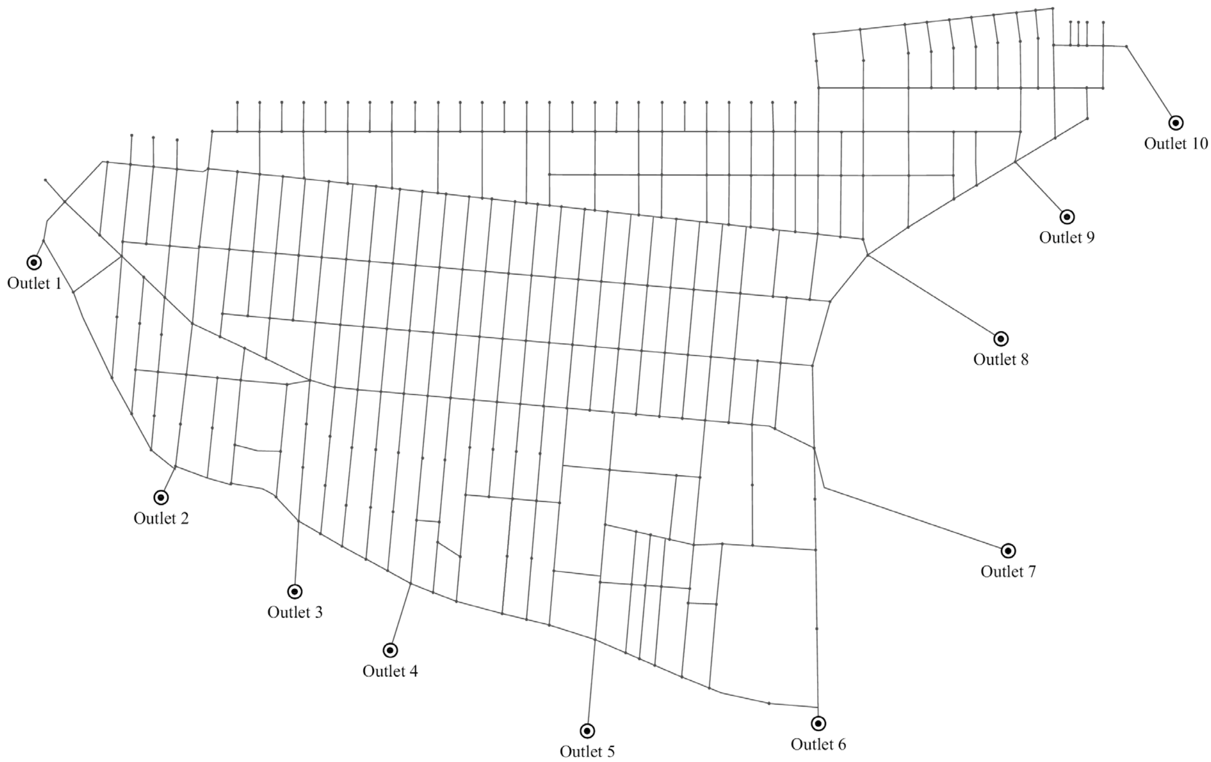


Figure 3.7: *Base graph of the case study*

The area under design has about 500 hectares of the highly urbanized area and a population of more than 100,000. Figure 3.7 presents the base graph of the case study. Based on the street map, it has 181 loops, 530 pipes (about 75 km length) and 10 candidate outlets. The Karun River, the largest river (average discharge of 575 m^3/s) in Iran, passes through the city and allows for stormwater to be drained at different locations. Water quality is not an issue here because of the short period of the rainy season, although treatment facilities could be considered in each outlet using the proposed method.

3.4.2 Sewer design

The next step for the sewer network with a given layout, generated by the *hanging gardens algorithm*, is to size the sewers and pumps. Sewer diameters and slopes, as well as the number and location of pumping stations, must be designed in a way that satisfies all hydraulic and technical constraints. The dynamic wave approach in SWMM is used with a 5-year design storm (6 hours duration with a total depth of 30.2 mm) for a corresponding hydraulic simulation. Surcharging is allowed to use the full capacity of the pipe networks, but flooding is not acceptable. The maximum allowable velocity in the pipes is 4 m/s, and the maximum allowable excavation depth is fixed at 5 m. Layouts that would need lift stations are automatically omitted using a penalty function. As the area is flat, the minimum allowable slopes from the Iranian manual are assigned to each pipe according to their diameters. Hence, the problem simplifies to determining the optimum size of pipes in the generated layout to minimize the costs while satisfying all hydraulic constraints. The design constraints, including minimum slopes for different diameters, are presented in Table 3.1: Design constraints for the case study. As the minimum required slope for smaller diameters is larger than for larger pipes, the optimum solution is implicitly related to both parameters. That means that the model sometimes prefers to use larger diameters in some branches to avoid deep excavation and the lowering of all other parts of the system.

Table 3.1: *Design constraints for the case study*

Description	Constraint
Maximum Velocity	4.0 m/s
Maximum excavation depth	5.0 m
Minimum cover depth	1.2 m
Minimum slope	0.0041 if D=200 mm
	0.0033 if D=250 mm
	0.0027 if D=350 mm
	0.0020 if D=400 mm
	0.0016 if D=500 mm
	0.0014 if D=630 mm
	0.0010 if D=800 mm
	0.0010 if D≥1000 mm

3.4.3 Optimization model

As our purpose is to demonstrate the *hanging gardens algorithm* and its capabilities to generate layouts with arbitrary degrees of centralization, it is sufficient to choose a single objective (minimum cost) problem. Likewise, the simplifications done above for the sewer design do not restrict the demonstration of the *hanging gardens algorithm*.

The number and location of outlets, the pipe layout in each part of the network and the diameters of each pipe are optimization variables. They are simultaneously optimized for minimum cost. This forms a hard class of combinatorial optimization which is nonlinear, mixed integer-real, highly constrained, large-scale and multimodal. A genetic algorithm is used as the optimization engine to find an initial solution. After that, Tabu search [52, 73] is employed to search for an optimum solution. The optimization starts with a randomly generated layout. Recall that the vectors α and β are sent to the *loop-by-loop cutting algorithm* to generate layouts in form of a centralized tree. The *hanging gardens algorithm* uses the generated layout and the vectors γ and ζ vector to determine the number and location of the outlets and to generate a forest of trees. Finally, a vector P is used to assign the diameter of pipes using equation 3.6 [55]:

$$D = D_{min} + (D_{max} - D_{min}) \times P \tag{3.6}$$

where D_{max} is the largest commercially available size and D_{min} is determined with respect to the telescopic pattern [89]. The diameter of every pipe must be equal to or greater than that of its upstream pipes, which means that:

$$D \geq max[DU] \tag{3.7}$$

where $[DU]$ contains pipe diameters connected to the upstream end of the pipe at hand. As the **H-matrix** saves the relation between the pipes hierarchically, $[DU]$ can be calculated simply for each pipe.

One of the most challenging tasks to design a sewer network in such flat areas is to satisfy the maximum excavation depth constraint and avoid too many lift pump stations, which are costly in terms of construction and operation. Therefore, the slope of every pipe, in this case, is set to the minimum allowable slope. However, slopes and pump stations can be considered in the proposed framework as trivial extensions. An SWMM input file is then created using the generated information, and the total cost of the current alternative is calculated. The life cycle costs (LCC) is used here as the objective function of optimization. The LCC evaluates the capital costs and the operation and maintenance (O&M) costs of the pipe network over a typical service period of 30 years [94]. The LCC of each alternative is calculated by compiling all the capital and O&M costs using Equation 3.8 to 3.10 to present-day [94]. The inflation rate of O&M cost in Iran is 12% and the discount rate of the total LCC is 15%. 10% of capital costs, from Iranian manual, are considered for annual O&M.

$$LCC = Capital_{sewer\ network} + PV_{O\&M\ sewer\ network} \tag{3.8}$$

$$PV_{\text{O\&M}} = \sum_{n=1}^{30} \text{Annual}_{\ O\&M} \frac{(1+r)^n}{(1+i)^n} \tag{3.9}$$

$PV_{O\&M}$ is the 30-year LCC for O&M of the sewer network, i is the discount rate, r is the inflation rate, and n is the years of service.

$$Capital_{sewer\ network} = \sum_{i=1}^{NP} CP_i + \sum_{i=1}^{NM} CM_i \tag{3.10}$$

in which CP and CM represent, respectively, the construction cost of sewers and manholes. These costs are estimated as a function of pipe diameter and buried depth using Table 3.2. NP and NM represent the number of pipes and number of manholes, respectively.

Hydraulic simulation is performed in SWMM. A penalty function is used to penalize any design that violates the hydraulic constraints. This procedure is iterated until it converges. Figure 3.8 illustrates this procedure. Table 3.3 summarizes the numbers and types of optimization variables.

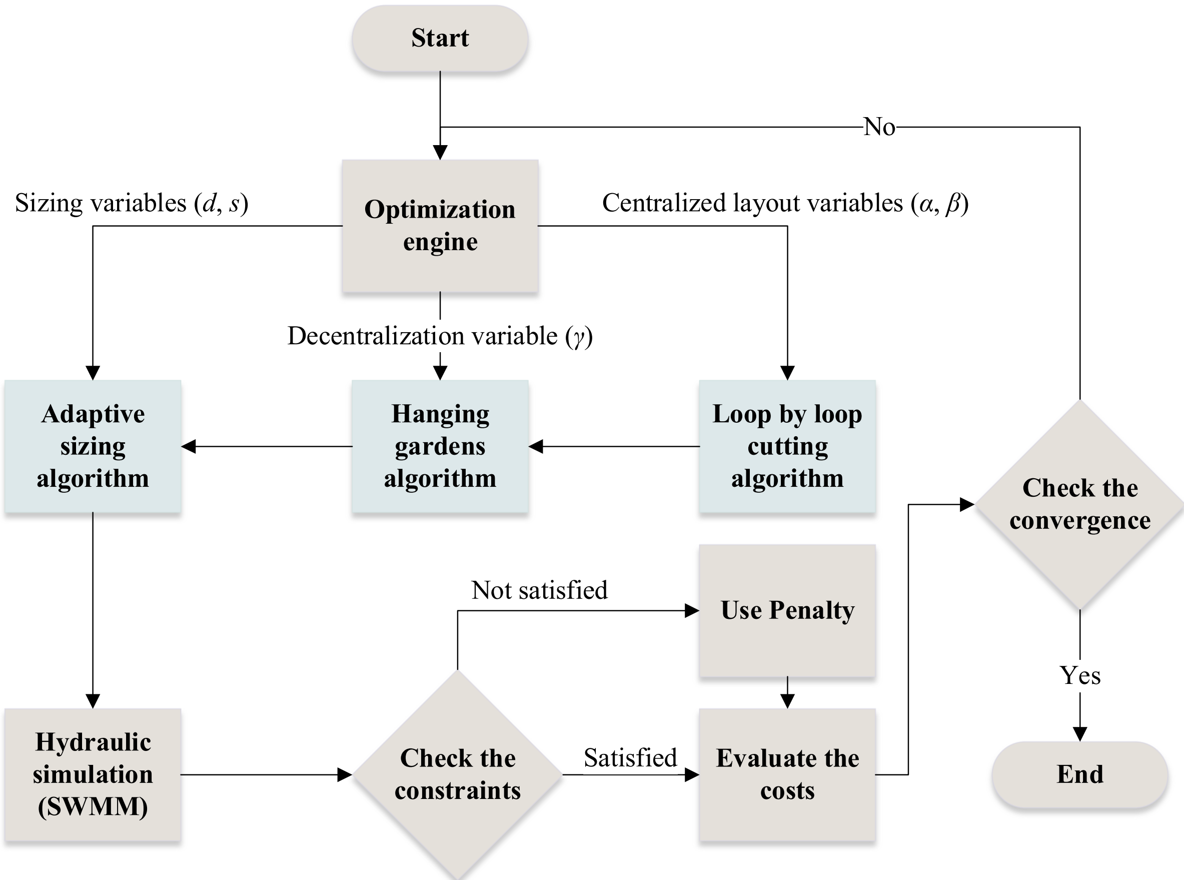


Figure 3.8: *Proposed framework for optimization of the case study*

Table 3.2: *Cost function of the case study*

Diameter (mm)	Cost of pipes (Million Rial/m)	Cost of manholes (Million Rial)
200	3.50H - 2.29	29.95H + 54.91
250	3.50H - 2.21	33.70H + 58.66
350	3.38H - 1.31	42.03H + 62.21
400	3.57H - 0.91	45.66H + 66.46
500	3.62H + 0.05	52.85H + 75.37
630	3.85H + 1.57	56.39H + 80.05
800	4.31H + 3.86	59.90H + 84.86
1000	4.65H + 8.12	73.65H + 105.56
1200	5.11H + 10.84	79.40H + 113.64
1500	5.73H + 15.55	91.20H + 129.86
2000	6.78H + 23.37	110.91H + 159.32

Note: H = average buried depth

Table 3.3: *Optimization variables of the case study*

Variable	Type	Number
α	real $\in [0,1]$	181
β	binary	181
γ	real $\in [0,1]$	10
ζ	binary	10
P	real $\in [0,1]$	530

3.5 Results and discussion

More than 100,000 layouts were generated and evaluated during the optimization using the hybrid GA-Tabu optimization approach. The computational time for solving the case study was about 72 hours using a personal laptop, Intel Core i7, with a 2.5 GHz dual-core CPU and 8 GB random access memory (RAM). The computation time for this case study is high because the hydraulic simulations are done based on the dynamic wave equations. Indeed, computation time in this order (~three days) is not that significant for the design and long-term planning of urban infrastructures.

Figure 3.9 presents the optimum design found by the proposed algorithm. The cost of this design is 205090 Million Rials. The maximum buried depth in the optimal design is 4.72 meters without any pumping station, which is desirable for the flat topography of the case study's area.

Seven out of ten candidate outlets are chosen by the optimization in the final layout. Using equation 4, the optimal DC obtained in this case study is 33.3%. Here, further decentralization leads to a diseconomy of scale, most likely because of the cost of long pipes needed to convey stormwater to the outlets. A systematic optimization across all those degrees and their possible combinations of used candidate outlets down to the scale of a fully decentralized layout would, up to date, not possible without the hanging garden algorithm.

Figure 3.10 shows the optimal fully decentralized design found by the proposed algorithm. Although no treatment facilities are considered in this case study, adding this kind of modules is trivial in the proposed framework. When considering the treatment facilities in this example, more centralized solutions are expected.

To compare the decentralized design with more centralized designs, the case study is also optimized with four outlets and only one outlet. Then, only the *location* of the outlet is considered as an optimization variable. Figure 3.11 and Figure 3.12 show the results of these optimizations. The cost of the centralized design (DC=100%) is about 30% more than that of the optimum decentralized solution. The maximum buried depth in this design is 6.78 m and the maximum pipe diameter is 2.0 m. For centralized design optimization, the constraint on maximum buried depth is neglected because no feasible solution would exist at all. Table 3.4 compares the design specifications of all optimized layouts.

Figure 3.9: *Final Design (DC=33%)*

Figure 3.10: *Optimal fully decentralized design (DC=0%)*

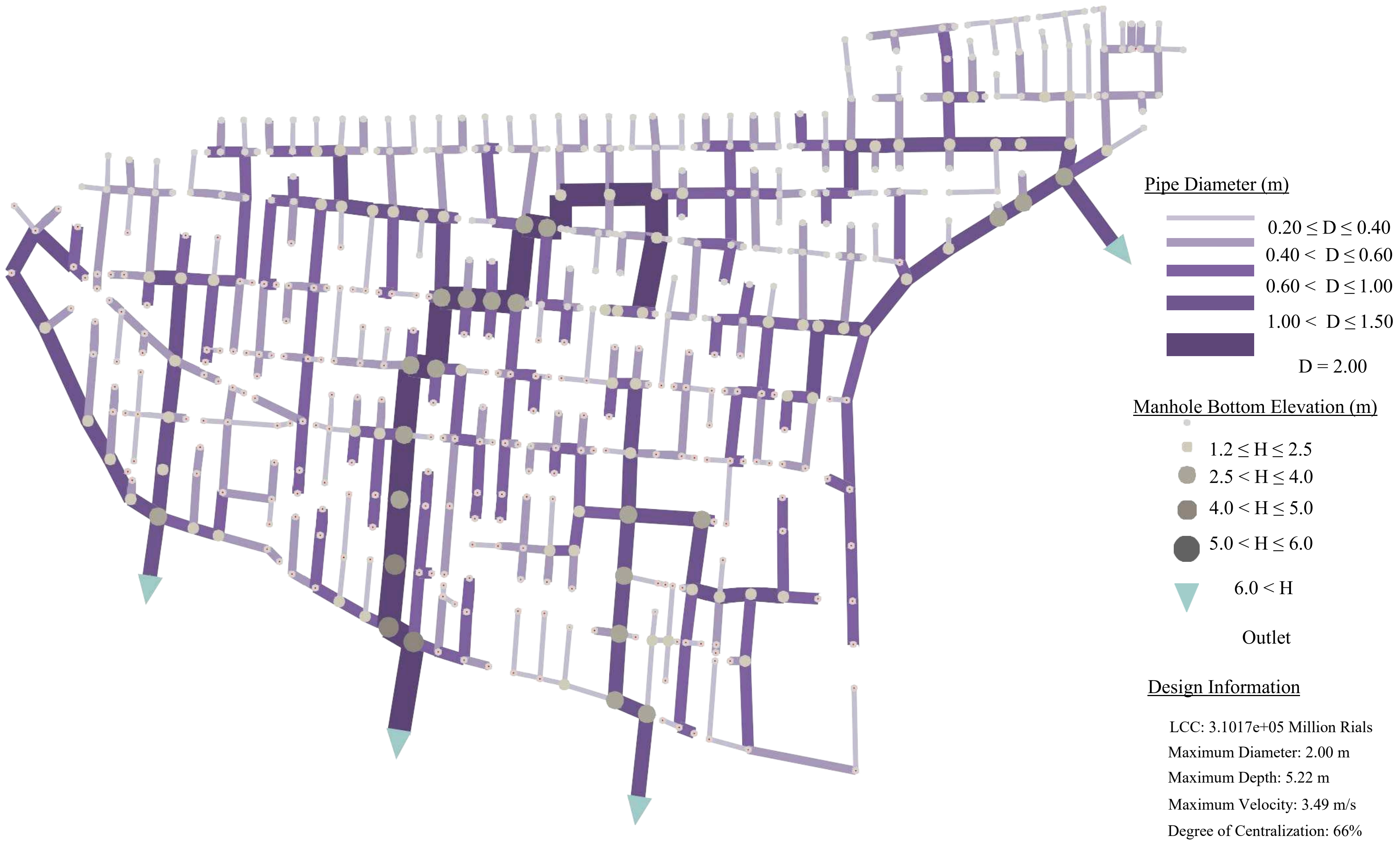


Figure 3.11: *Optimal design (DC=66%)*

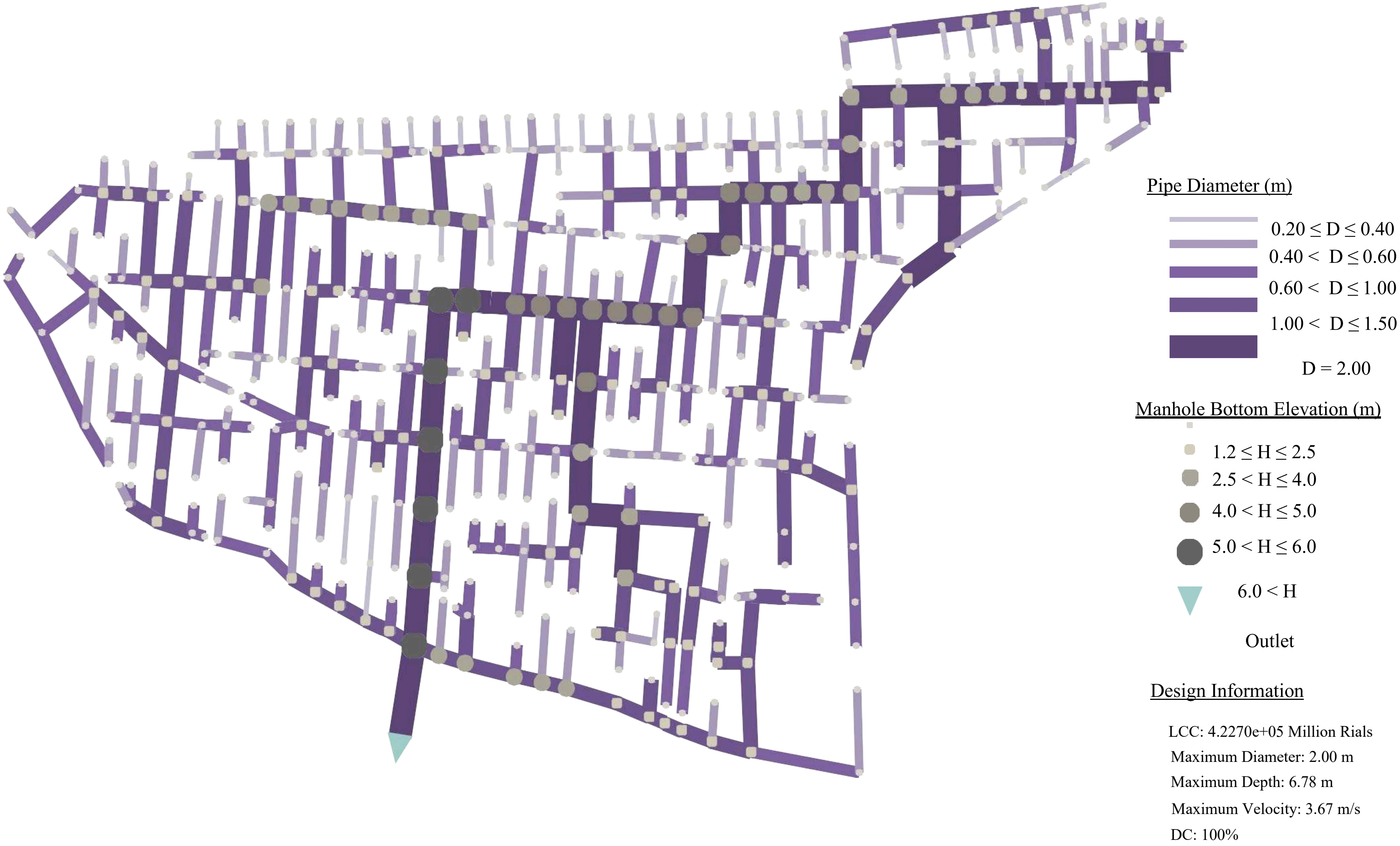


Figure 3.12: *Optimal fully centralized design (DC=100%)*

Table 3.4: *Comparison between design specifications of the three optimal layouts*

Design	LCC (M. Rials)	Cap. Gray	O&M Gray	Average Diameter (m/m)	Average buried depth (m)	Maximum Diameter (m)	Maximum buried depth (m)
100	422700	138860	283840	0.88	2.57	2	6.78
66	310170	101900	208270	0.68	2.01	2	5.22
33	250953	82433	168520	0.64	2.13	1.5	4.72
0	253197	83177	170020	0.63	2.16	1.5	4.98

Besides the economic concerns, as discussed in the introduction, there are other drivers for reducing the degrees of centralization, like increasing the system resilience and reducing the risk of failure. The resilience of each optimized scheme with different degrees of centralization is assessed with the hydraulic performance indicator (HPI) using equation 9 adopted from [95]. For this purpose, the design storms with the return periods of 10, 20, 25 and 50 years (6 hours duration) and total depths of 38.3, 46.7, 49.5 and 58.5 mm, respectively, are used. The HPI is defined by:

$$HPI = 100 \times \left(1 - \frac{V_{flooding}}{V_{runoff}}\right) \qquad (\%) \tag{3.11}$$

Here, $V_{flooding}$ is the total water that overflows the nodes, and V_{runoff} is the total runoff volume. The results of this analysis are presented in Table 3.5. As seen in this table, the centralized layout exhibits a slightly better performance during a 10-year storm. However, the performance alters in favor of the two decentralized alternatives for the heavier storms because of the vulnerability of the single main collector in the centralized design. Although the centralized system has, on average, an additional storage capacity, all upstream parts of its main collector are affected as soon as that main collector encounters its maximum capacity. Contrariwise, the decentralized systems provide different alternatives for drainage, and the performance of each part does not disturb other parts. Here, the scenario with DC = 66.6% has the most resilience plausibly because it takes the advantages of both having several drainage alternatives and more storage capacity. Although not analyzed here, it is clear that this trend remains the same for hazards like aging infrastructures, pipe blockages or breaks in the pipes.

Table 3.5: *Comparison between hydraulic performances of the four optimal layouts during different design storms*

DC (%)	5 years (30.2 mm)	10 years (38.3 mm)	20 years (46.7 mm)	25 years (49.5 mm)	50 years (58.5 mm)
100	22	97.83	88.28	85.11	76.91
66.6	11	98.80	95.75	94.42	89.05
33.3	11	96.97	91.85	90.01	84.43
0.00	33	97.26	91.93	90.07	84.49

The other key driver for hybrid urban drainage systems is their adaptability for forthcoming challenges. In the future, existing urban drainage systems should be improved to meet more stringent regulations to endure climate change effects on intensity and duration of storms and to show an apt response to city growth and changing land use [13]. Changing a piecewise decentralized system to account for these challenges is much easier and more cost-effective than altering a centralized system with larger pipes and higher buried depths.

3.6 Conclusion

Conventional centralized urban drainage systems are costly to construct, maintain and operate. They have limited resilience and adaptiveness to upcoming challenges like climate change, rapid urbanization, aging infrastructures and rising environmental-ecological standards. On the other hand, constructing fully decentralized systems is neither practical nor economical. Therefore, finding the optimal degrees of centralization among the multitude of alternatives is a challenging and vital task.

A brief overview of the literature shows that only a few approaches are available for generating and optimizing decentralized urban drainage alternatives, which are still far from real applications. To fill this gap, a layout generator, namely the *hanging gardens algorithm*, was developed in this study to generate all possible hybrid urban drainage systems for both flat and steep terrains.

To generate a decentralized layout, a random centralized layout is firstly generated using the *loop-by-loop cutting algorithm*. Then, the *hanging gardens algorithm* picks an arbitrary combination of outlets from a pre-defined candidates list. Next, it adds outlets from the selected set to the generated layout and uses a graph-theory based approach to assign parts of the layout to different outlets. This procedure is iterated until all outlets on the list have been included. To form a simulation-optimization framework, an optimization engine is coupled with the proposed layout generator algorithm and with hydraulic simulation software. The model was then applied against a real case study. The proposed model exhibited good performance in exploring different degrees of centralization, generating realistic layouts and finding near-optimum solutions. Since the mathematical representation of generated networks is close to that of real sewer systems, the proposed framework introduces a comprehensive design package that can be employed for more realistic design as a superiority to existing conceptual models.

The *hanging gardens algorithm* works on a random base and is self-adaptive so that any set of arbitrary decision variables $(\alpha, \beta, \gamma, \zeta)$, always lead to a feasible layout. Therefore, it can be coupled with any unconstrained metaheuristic as well as hydraulic simulation software. The algorithm can be used for designing both new and existing networks. To do the latter, (1) the existing pipes should not be involved in any loop of the base graph and (2) they should be fixed in the optimization in the design step. It is also possible to let the optimization model decide whether it is better to change an existing part in the system or to keep it as it is.

In the future, the proposed framework could be extended to consider other decentralization measures like green infrastructures for stormwater management or different treatment facilities for sewage collection or combined systems. The proposed algorithm has no restrictions that would hinder such extensions. Furthermore, the relationship between other design objectives like reliability, resilience, vulnerability and the degrees of centralization needs to be explored in a multi-objective framework. Besides, the uncertainties for both the design parameters and future hazards (e.g. climate change and dynamic of cities) and their effect on the optimal degrees of centralization could be investigated.

Chapter 4. Hybrid decentralized green-blue-gray UDSs optimization

Most of the content of this chapter has been published in the *Journal of Environmental Management* under the title "Hybrid green-blue-gray decentralized urban drainage systems design, a simulation-optimization framework" [42].

Summary

Recent studies suggested hybrid green-blue-gray infrastructures (HGBGI) as the most promising urban drainage systems that can simultaneously combine reliability, resilience and acceptability of gray infrastructures (networks of pipes) with multi-functionality, sustainability and adaptability of green-blue infrastructures (GBI). Combining GBI and gray measures for designing new urban drainage systems forms a nonlinear multimodal mixed integer-real optimization problem that is highly constrained and intractable. For this purpose, this chapter presents a simulation-optimization framework to optimize urban drainage systems considering HGBGI alternatives and different degrees of centralization. The proposed framework begins with the characterization of the site under design and drawing the base graph. Then, different layouts with different degrees of centralization are generated and hydraulically designed using the *hanging gardens algorithm* developed in Chapter 3. After introducing the feasible GBI to the model, the second optimization is performed to find the optimum distribution of GBIs in a way that minimizes the total life cycle costs of GBIs and pipe networks. Finally, the resiliency and sustainability of different scenarios are evaluated using several design storms that provide material for final assessment and decision-making. The performance of the proposed framework is evaluated again using Ahvaz test case introduced in Chapter 3.

4.1 Introduction

Recent research castigates the performance of traditional urban drainage systems (UDS) that are based on only gray infrastructures (e.g. pipe networks, storage tanks and centralized WWTPs) in coping with upcoming challenges such as climate change, urban growth and providing long-term sustainability [5, 6]. Traditionally, different gray infrastructures are chosen based on economic efficiency and local conditions for urban water management. This approach is based on the collection and fast transfer of runoff, which results in many adverse impacts on the environment. Hydrological disruption, groundwater depletion, downstream flooding, pollution in water bodies, and stream ecosystem damage are a sample of degrading legacies of gray infrastructures [11, 12].

Nowadays, it is becoming a well-accepted fact that other objectives such as socio-ecological sustainability, resilience and adaptability needs to be considered in the planning or rehabilitation phase of urban water infrastructures [26, 29, 31, 32]. Therefore, various sustainable stormwater management measures have been recommended to mitigate the aforementioned problems in more environmentally-friendly ways [20, 46–48].

These multi-functional and decentralized (distributed) measures are generally referred to as low-impact development, best management practices, green infrastructures, green-blue infrastructures (GBI), water sensitive urban design, etc [20, 49, 50]. Notwithstanding, the distinct terminologies differ lightly in their meanings due to their histories [49]. In this chapter, I use the term GBI. Some common GBI practices are bio-retention cells, infiltration trenches, stormwater wetlands, wet ponds, permeable pavements, swales, green roofs, filter strips, sand and gravel filters and rain barrels [11, 20, 51].

For the optimal selection of type, location and size of GBIs, numerous optimization and decision-making tools and methods, as reviewed in the next section, have been developed so far in the literature.

The focus of methods to optimize GBIs is mainly finding optimum retrofitting strategies through combing GBIs with existing gray infrastructures [26]. Although the advantages of including GBIs for retrofitting purposes have been widely discussed and acknowledged, mainly in the developed countries, they cannot fully replace conventional gray infrastructures especially in developing countries and for planning new infrastructures [25]. The reasons are lack of space in the highly urbanized areas, socio-economic factors, the lack of environmental awareness and public acceptance and GBI's inability to control extreme events [5, 11]. To conclude, gray measures are largely tested systems that show more resilience to cope with intense rainfall while GBIs offer multiple benefits such as adaptability and sustainability [26, 27].

Many authors and studies so far have suggested hybrid green-blue-gray infrastructures (HGBGIs) as the most promising urban water management approach that can simultaneously combine reliability, resilience and acceptability of conventional pipe networks with multi-functionality, sustainability and adaptability of green [26, 29]. Combining GBIs and gray measures makes the procedure of designing urban water infrastructures more complicated, as there are many feasible combination scenarios. Therefore, there is a need for developing new methodologies and tools to facilitate the combined optimization process. This chapter aims to present a simulation-optimization framework to optimize UDSs considering HGBGIs and different degrees of centralization. The review of previous studies in the upcoming section will reveal that there is no tool or methodology for identifying the effects of the interaction between gray and green-blue infrastructures in the design phase of UDSs.

The remainder of this chapter is structured as follows: in the next section, the state of the literature on HGBGI optimization is recapitulated and the research gaps in the field are identified. In the material and methods section, the proposed framework is presented in detail. To demonstrate the performance of the proposed framework, it is applied against the case study. The case study features the city of Ahvaz in Iran. The results are presented and scrutinized in the results and discussion section. The last section concludes the chapter and provides recommendations for further investigations.

4.2 Literature review

The optimal planning and design of GBIs is a complex task as their design is bounded with various purposes and objectives, choosing among different types, design parameters and considering their spatial allocation [19, 22, 94, 96].

Among many hydro-environmental objectives of using GBIs are water quality improvement, water quantity reduction, flood mitigation, recharging groundwater, water harvesting, restoring the hydrologic characteristics of the site, increasing urban amenity and alleviating the urban heat island effect [11, 21–23, 97–99]. The design decisions include the size, type, number and location of components and how they are connected. This issue needs to satisfy several practical constraints including cost, space availability and site characteristics, including soil type, topography, infiltration rate, contributing connected impervious area and restrictions due to regional plans or legal regulations [22, 100, 101].

Some studies combined hydrologic and hydraulic simulations with optimization techniques to identify the optimal or near-optimal selection, sizing and location of GBIs. A few works have combined the hydrodynamic models with multi-objective optimization to evaluate and compare different configurations of green-blue-gray practices and their effects [5]. Table 4.1 summarizes some studies that used optimization techniques for designing GBIs or HGBGI. The criteria specified in the table for each study are (1) whether they considered gray practices (layout of the network or hydraulic specification), (2) whether they are aimed at selection, sizing or determining the location of GBIs and (3) the type of objective(s) considered (e.g., cost, peak flow reduction, water quality improvement, other performance indices like reliability and resilience).

Among the methods presented in Table 4.1, only four cases comprise of a combination of GBIs and gray practices (i.e., HGBGI). Damodaram and Zechman (2013) [29] developed a simulation-optimization framework to identify and explore watershed management plans that utilize green strategies (permeable pavement and rainwater harvesting) and gray strategies (detention ponds) to reduce the impacts of peak flow by a range of design storms for varying budget levels. They found that LID/BMP hybrids performed the best, but that the peak flow metrics might not be the best for judging sustainability. Alves et al. (2016) [5] presented a multi-objective optimization framework to select, evaluate and place different green-gray practices for retrofitting UDSs. The proposed approach was applied to a highly urbanized watershed to evaluate the effects of green-gray infrastructure (green roof, infiltration trench, permeable pavement and storage tank) on reducing the quantity of combined sewer overflow. They concluded that the lack of space faced in the highly urbanized areas, where drainage systems have to be enlarged, can be confronted if centralized and distributed practices are combined. Duan et al., (2016) [102] studied a multi-objective optimal design of gray practices (detention tanks) and green practices (bio-detention tank, rain garden, permeable pavement and green roof). They reported that both total investment costs and flooding risk could be significantly reduced by optimally designed detention tanks and GBI measures.

All of the approaches reviewed above, only comprised storage/detention tanks as gray practices to couple with GBIs. To the best of our knowledge, no study so far has investigated the effect of coupling GBIs with conventional pipe networks in the planning phase of stormwater management networks. Therefore, the main aim of this chapter is to develop a simulation-optimization framework to design hybrid green-blue-gray stormwater management systems to answer the following questions:

1- Can HGBGIs compete with conventional gray infrastructures economically?

2- How do the layout configuration of pipe networks and the degree of centralization affect the economic efficiency of HGBGIs?

3- How do HGBGIs compare with gray systems in terms of resilience and sustainability?

Table 4.1: *Analysis of optimization studies for decentralized stormwater management systems*

	Gray measures		GBI			objective/s			
	Layout	Hydraulic	Type	location	size	Cost	Quality	quantity	Performance
Chapter 4	✓	✓	✓	✓	×	✓	×	×	✓[a]
Giacomoni and Joseph (2017) [101]	×	×	✓	✓	×	✓	×	✓	✓[b]
Di Matteo et al. (2017) [100]	×	×	✓	✓	✓	✓	✓	✓	×
Chui et al. (2016) [94]	×	×	✓	×	✓	✓	×	✓	×
Lee et al. (2012) [103]	×	×	✓	✓	×	✓	✓	✓	×
Jia et al. (2015) [27]	×	×	×	×	✓	✓	✓	✓	×
Duan et al. (2016) [102]	×	✓	✓	×	✓	✓	×	✓	×
Baek et al. (2015) [104]	×	×	✓	×	✓	×	✓	×	×
Cano and Bark-doll (2017) [21]	×	×	✓	✓	×	✓	✓	×	✓[c]
Liu et al. (2016) [105]	×	×	✓	✓	×	✓	✓	✓	×
Sebti et al. (2016) [106]	×	×	✓	✓	×	✓	✓	✓	×
Li et al. (2018) [99]	×	×	✓	✓	×	✓	✓	✓	×
Alves et al. (2016) [5]	×	✓	✓	✓	×	✓	×	✓	×
Stafford et al. (2015) [107]	×	✓	×	×	✓	✓	×	×	×
Oraei Zare et al. (2012) [108]	×	×	×	✓	✓	✓	✓	✓	×
Jayasooriya et al. (2016) [109]	×	×	×	×	✓	✓	✓	×	×
Dandy et al. (2018) [22]	×	×	✓	✓	✓	✓	✓	✓	✓[d]
Kaini et al. (2012) [110]	×	×	✓	✓	✓	✓	✓	×	×
Damodaram and Zechman (2013) [29]	×	✓	×	✓	×	✓	✓	×	×
Zhang et al. (2013) [111]	×	×	✓	✓	✓	✓	✓	×	×

[a] Resilience and sustainability; [b] Hydrologic footprint residence; [c] Maintenance probability factor; [d] Volumetric reliability

4.3 Materials and methods

4.3.1 Problem formulation

This chapter aims to investigate the performance of HGBGIs considering different degrees of centralization. In general, the mathematical least-cost optimization of sewer networks is formulated as follows.

$$\boldsymbol{d}_{\text{opt}} = \arg \min_{\boldsymbol{d} \in \boldsymbol{D}} [f_{\text{cost}}] \tag{4.1}$$

where $\boldsymbol{d}_{\text{opt}}$ is the optimal choice for the decision variable $\boldsymbol{d}$ in the feasible space $\boldsymbol{D}$ that defines the sewer system. It includes the degree of centralization, layout configuration and hydraulic specifications. By adding the GBIs as new variables to the problem, $\boldsymbol{d}$ could be extended as the following.

$$\boldsymbol{d} = [DC, layout\ parameters, hydraulic\ parameters, GIB\ parameters] \tag{4.2}$$

where DC (degree of centralization) implicitly explains how the system as a whole is distributed as defined in the layout generation section. Layout parameters express the connectivity between different components (sewers and pumping stations) connected to one outlet each in a way that satisfies all layout constraints. Hydraulic parameters determine pipe diameters, pipe slopes and pump stations to satisfy all hydraulic and technical constraints. GBI parameters involve the type, size and location of GBIs. GBIs can help to reduce the size and consequently the cost of pipe network in two main ways:

1. **By peak flow reduction** due to capturing stormwater and altering concentration time, the size of the pipe network can be reduced.
2. **By replacing the pipes in the upstream branches**, GBIs might capture all stormwater for the designed rainstorm. Also, removing pipes in the upstream branches reduces the installation depth of pipes further downstream and would lead to a cost reduction. However, construction and maintenance of GBIs have costs. Therefore, the optimization will find the optimum mixture of HGBGIs.

Considering all variables declared in Equation 2 for Equation 1 forms a nonlinear multimodal mixed integer-real optimization problem that is highly constrained and large-scale in most of the real cases. To solve this hard-class combinatorial optimization problem and finding a near-optimum solution in a reasonable time, this study presents a simulation-optimization framework. To keep computation costs at a plausible level, and to obtain directly pairwise comparable results between gray and hybrid systems, I split the joint optimization into two steps. First, different sewer layouts with different DCs from fully centralized to fully decentralized are generated and hydraulically designed for the least cost using the *hanging gardens algorithm* developed in Chapter 3. Then, the feasible GBIs are introduced to the model and a second optimization is performed on each of the layouts to find the optimum distribution of GBIs in a way that minimizes the total life cycle costs (LCC) of GBIs and the pipe network. Finally, to choose between the optimal solutions with different DC, the resilience and sustainability of different scenarios are evaluated using several design storms that provide material for decision-making.

The proposed framework is shown schematically in Figure 4.1 The details of all sub-processes and algorithms are given in the following sections. It must be noticed that none of the obtained designs using the proposed framework are global optimum, though the results can be used for rough comparison and decision-making.

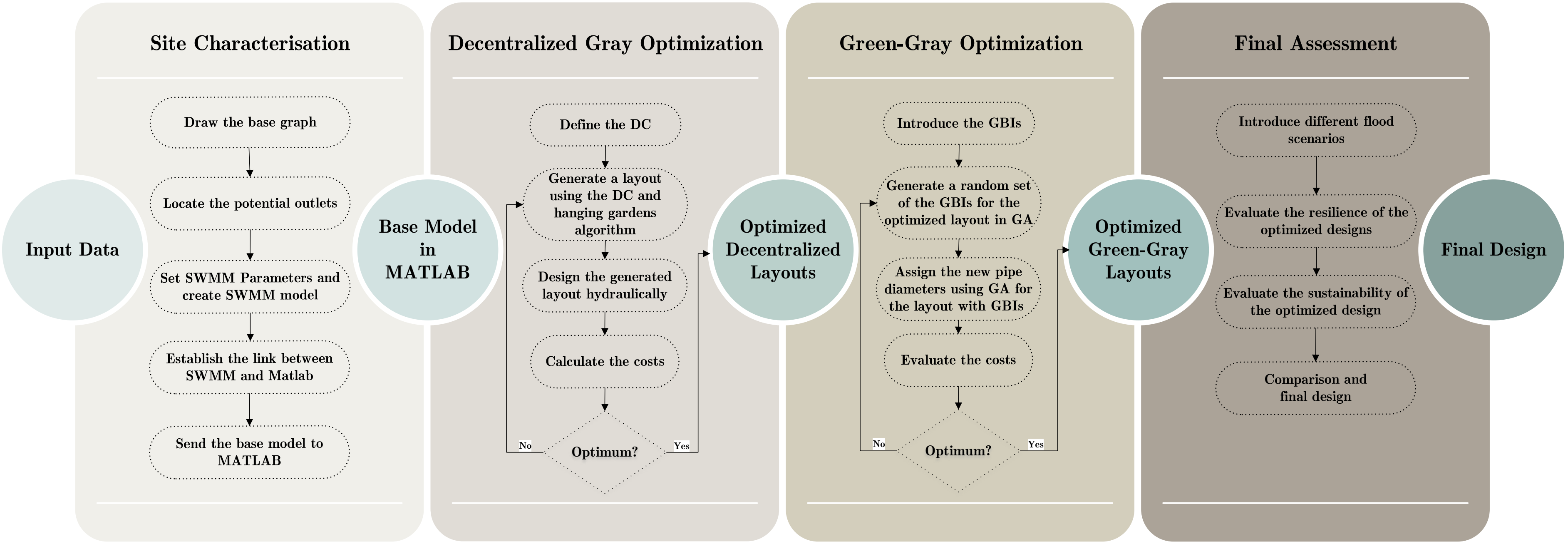


Figure 4.1: *The proposed framework*

4.3.2 Step 1: Site characterization

Case study

To show how the proposed framework works, Ahvaz case study that already has been introduced in Chapter 3 is used. As stated in Chapter 3, the case study is a part of the city of Ahvaz located in the southwest of Iran. It has a semi-desert climate with long and very hot summers and short and mild winters. Annually, urban flooding due to lack of a stormwater management system causes public inconvenience, economic and environmental destruction. The area under design is located in a highly urbanized area with flat topography and relatively high groundwater level. Technically, the aforementioned issues make constructing a conventional pipe network with large pipe diameters and deep excavations too expensive and almost impossible in practice.

GBI selection

Possible GBI options include green roofs, rain gardens, swales, permeable pavements, infiltration trenches, ponds and rain barrels. The main factors to select candidate options for the upcoming optimization include (1) the design objectives such as runoff/peak flow reductions and quality control, (2) site characteristics such as topography, degree of urbanization, climate and social concerns and (3) costs [19].

Through the following paragraphs, literature in this area is reviewed. Based on these suggestions, I will choose appropriate GBI candidates and their specifications to be added during optimization.

First, I review the influence of design objectives. Damodaram et al. (2010) [28] reported that infiltration-based GBI measures are more effective than storage-based measures for smaller storms (18 mm, 45 mm) but that storage-based measures are more effective for larger storms (114mm, 185mm, 279mm). Baek et al. (2015) [104] found that bio-retention and rain barrels are most effective for reducing the first flush effect of suspended solids. Li et al. (2017) [24] concluded that the preferential order of GBI single measures is: bio-retention > rain barrels > low-elevation greenbelts > green roofs > permeable pavement. Zhang and Chui (2018) [20] suggested that combining diverse GBI practices can improve system functionality. As an example, they reported that infiltration-based GBIs like infiltration trenches combined with storage-based ones such as bio-retention cells and rain barrels can lead to better stormwater management by providing different approaches for rainfall-runoff control.

Second, I review how GBI solutions for stormwater management depend on location characteristics like soil type, rainfall patterns and land use types as they generally rely on infiltration and evapotranspiration [20]. The design of GBI stormwater strategies and controls must consider such site-specific conditions to be successful [19]. Bloom (2006) [112] suggested some detention-based management methods for flat areas.

Third, I consider the costs. Stovin and Swan (2007) [113] ranked LID measures based on their costs from the least to the most expensive as follows: infiltration basins, soakaways, ponds, infiltration trenches and porous pavement. Joksimovic and Alam (2014) [114] showed that infiltration trenches and a combination of infiltration trenches with green roofs are the most cost-efficient solutions for runoff reduction. Another important decision variable that influences the cost-effectiveness is the size of the GBI measure. Zhang and Chui (2018) [20], recommended that the area of the bioretention cells should be 8%–25% of the drainage area and that one have an area-to-depth ratio for bioretention cell, of 50 cm - 120 cm. Chui et al. (2016) [94], recommended expanding bio-retention cells and porous pavements in the area instead of increasing in depth, whereas green roofs are recommended to increase in depth instead of expanding in the area.

As discussed earlier, the current case study is located in a highly urbanized and flat terrain, and the design storm is about 31 mm. Based on the above recommendations, and according to the main objective of the problem (cost optimization), I select rain barrels and infiltration trenches as GBI options. Rain barrels are micro-scale GBIs that are used as temporary storage and for rainwater harvesting [115]. Infiltration trenches are buried storage units filled with drain rock that have a significant amount of underground storage. Therefore, infiltration trenches are useful for areas with limited space [116]. After rainfall, runoff from the roofs is diverted into rain barrels to supply water for toilet flushing and household irrigation. A percent of the impervious area from roads and parking lots, and roof runoff overflowing the rain barrels are diverted to infiltration trenches. It is supposed that each apartment can be equipped with a 2 m^3 rain barrel that is available in the local market. The infiltration trenches are installed along streetscapes and can cover on average up to 5% of the impervious area in each sub-catchment. Each Infiltration trench unit is supposed to have 2 m width, 5 m length and a berm height of 250 mm. Other design parameters are assigned or estimated according to the literature as follows: vegetation volume fraction 0.1, storage (gravel) layer thickness of 1500 mm, a void ratio of 0.75, seepage rate of 0.56 mm/h, drain flow exponent of 0.5, and an offset height of 100 mm [21, 94, 116]. This specification removes the selection of GBI type and size from the optimization, and only the locations for GBIs remain to be decided.

Life cycle costs (LCC)

As the objective function in Equation 1, I use life cycle costs (LCC). Recall from Chapter 3, the LCC evaluates the capital costs and the operation and maintenance (O&M) costs of the pipe network and implemented GBIs over a typical service period of 30 years [94]. The LCC of each alternative is calculated by compiling all the capital and O&M costs using Equation 4.3 and 4.4 to present-day [94]. The inflation rate of O&M costs in Iran is 12% and the discount rate of the total LCC is 15%. The construction cost of the pipe network is given in Chapter 3 (equation 3.10), 10% of capital costs, from the Iranian manual, are considered for annual O&M. To estimate the construction costs of infiltration trenches, the cost of excavation, removal, dewatering, grading, geotextile fabric, underdrain pipe and drain rock has been considered [116]. The initial construction cost of each infiltration unit is calculated as 113.4 M. Rials and 5% of capital costs are considered for annual O&M [111]. The price for each rain barrel in the local market is 19.423 M. Rials.

$$LCC = Capital_{green-blue} + PV_{\text{O\&M}\,green-blue} + Capital_{gray} + \text{PV}_{\text{O\&M}\,gray} \tag{4.3}$$

$$PV_{\text{O\&M}} = \sum_{n=1}^{30} \text{Annual}_{\,O\&M} \frac{(1+r)^n}{(1+i)^n} \tag{4.4}$$

$PV_{O\&M}$ is the 30-year LCC for O&M of green-blue or gray infrastructures, i is the discount rate, r is the inflation rate, and n is the years of service.

4.3.3 Step 2: Decentralized gray optimization

To design a conventional gray sewer system as the first step, a feasible layout considering street alignments, topology, barriers, watercourses and locations of the outlets is designed. Second, the hydraulic specifications of the generated layout are designed. To generate a sewer layout with an arbitrary DC, the *hanging garden algorithm* introduced in Chapter 3 is adopted. For this purpose, several outlet candidates are nominated in the area and a centralized layout with an arbitrary outlet is generated. Then, other arbitrary outlets from the candidates are added to the generated layout considering the desired DC. For the generated layout, pipe diameters and invert elevation are designed in a way that satisfies all hydraulic and technical constraints. To satisfy technical constraints like the telescopic pattern, minimum cover depth, maximum excavation depth and minimum and maximum slope, the adaptive approach introduced in Chapter 3 is used.

4.3.4 Step 3: Hybrid green-blue-gray optimization

This section introduces the proposed simulation-optimization framework to consider HGBGIs for urban water management with arbitrary DC. After generating and optimizing UDSs with different DCs as explained in section 4.3.3, a list of feasible GBIs is added to the optimization problem, selected according to section 4.3.2.

The locations of GBIs are considered as optimization variables. Therefore, for each sub-catchment, there is a binary variable so that 1 means the sub-catchment is equipped with GBI and 0 means the sub-catchment has no GBI. By adding a GBI, the size of pipes could be reduced by optimization. To consider this effect and to lighten the search space to perform the optimization in a reasonable time, 4 alternatives for each pipe are considered here: The diameter of a pipe can remain the same as the optimum gray design or it can be reduced down to three smaller sizes from the available commercial pipe sizes. For example, a pipe with 1.50 m diameter in an optimized gray design can have 1.5, 1.2, 1.0 and 0.8 m diameter and a pipe that already has 0.35 m diameter can have 0.35, 0.25, 0.20 m diameter or be removed from the network as there is no smaller pipe in the list. To meet the telescopic pattern constraint, the minimum diameter that can be assigned to a pipe is restricted by the diameter of its upstream pipe.

For the optimization, a simple binary genetic algorithm (GA) is developed. In a binary GA, real decision variables are encoded with binary 0-1 values (bits). Each chromosome represents a design alternative. Here, there are $NP + NS$ variables including NP pipe diameters, and NS GBI indicators in each sub-catchment. To get the least-cost design, these variables need to be calibrated by the GA to minimize the LCC in Equation 4.3. Considering two binary bits to represent each pipe diameter parameter and one bit for GBI in each sub-catchment parameter, a design chromosome is consisting of $2NP + NS$ genes (0-1 values) as shown in Figure 4.2. For each chromosome, sewer diameters D and GBI indicators are decoded as shown in equations 4.5 and 4.6. Figure 4.3 presents this approach schematically. This optimization is performed for each optimal gray-only system corresponding to the different degrees of centralization.

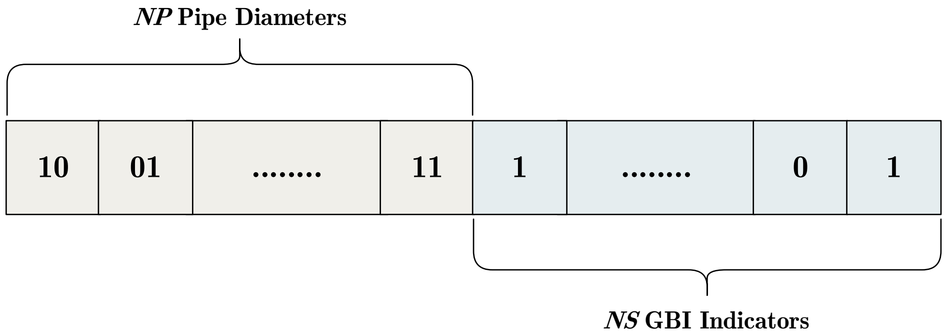


Figure 4.2: *Binary chromosome of a randomly generated design alternative*

$$d = \begin{cases} 11 \rightarrow D = Same\ as\ in\ the\ gray\ design \\ 10 \rightarrow D = One\ size\ smaller\ than\ in\ the\ gray\ design \\ 01 \rightarrow D = two\ size\ smaller\ than\ in\ the\ gray\ design \\ 00 \rightarrow D = three\ size\ smaller\ than\ in\ the\ gray\ design \end{cases} \tag{4.5}$$

$$GBI = \begin{cases} 1 \rightarrow Subcatchment\ has\ \mathrm{GBI} \\ 0 \rightarrow Subcatchment\ has\ no\ \mathrm{GBI} \end{cases} \tag{4.6}$$

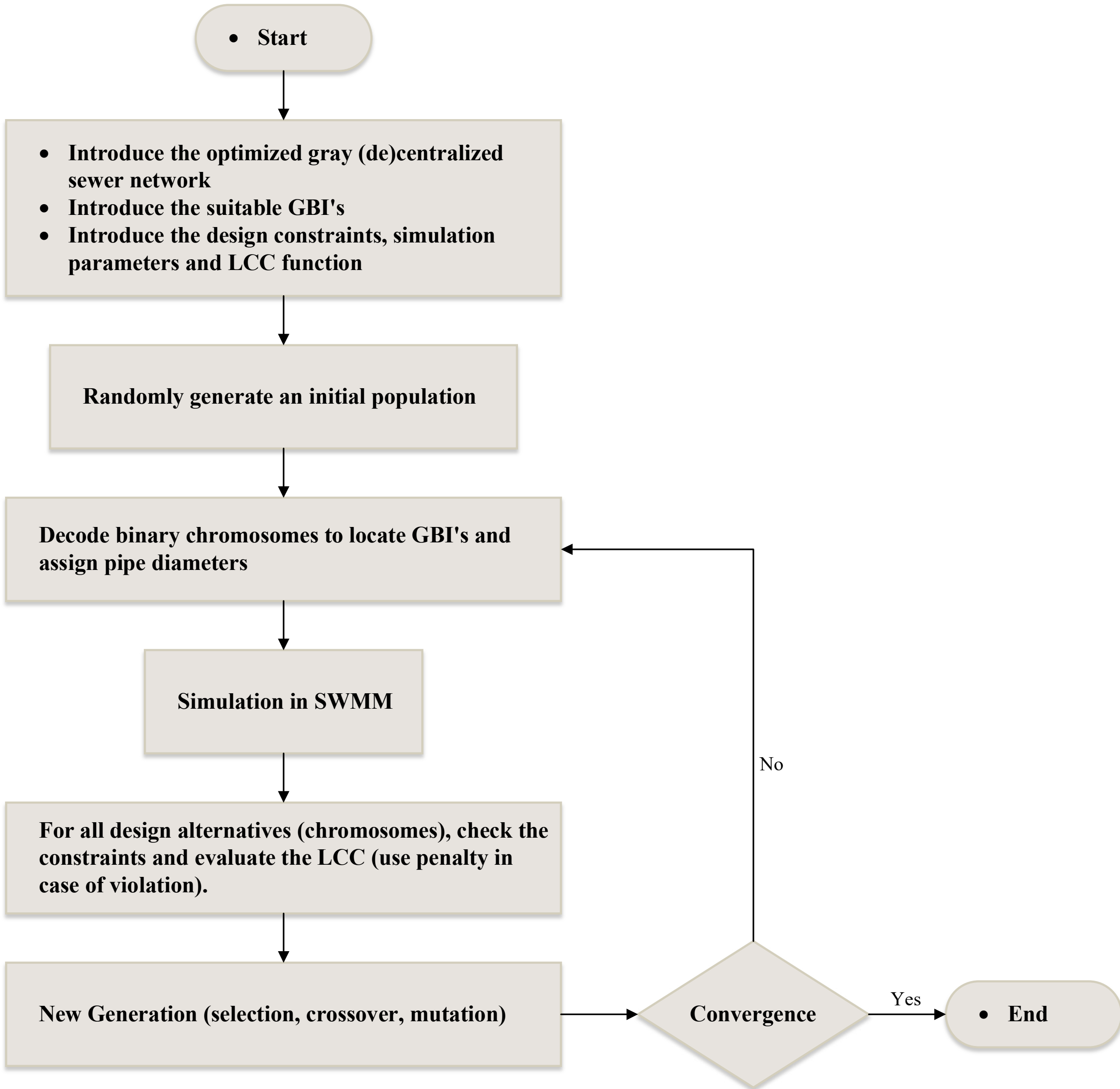


Figure 4.3: *Hybrid green-blue-gray optimization flowchart*

4.3.5 Step 4: Final assessment

To help the process of decision making among the remaining alternative DCs, at the final step, other crucial criteria not considered during the optimization are taken into account. For the case study, two simple indices as indicators for resilience and sustainability are defined. By definition adopted from Butler et al. (2014) [35], the resilience is "the degree to which the system minimizes the level of service failure magnitude and duration over its design life subject to exceptional conditions". The resilience of each optimized scheme with different degrees of centralization is evaluated with the hydraulic performance indicator (HPI) using equation 4.7 [95].

$$HPI = 100 \times \left(1 - \frac{V_{flooding}}{V_{runoff}}\right) \qquad (\%) \tag{4.7}$$

where $V_{flooding}$ is the total water that overflows the nodes, and V_{runoff} is the total runoff volume.

Butler et al. (2014) [35], defined sustainability as "the degree to which the system maintains levels of service in the long-term whilst maximizing social, economic and environmental goals". As a simple index for environmental sustainability, the ratio between the storage quantity and total precipitation under storm design is used in this study (Equation 4.8).

$$SUS_{Env} = \frac{infiltration\ volume + \ final\ GBI\ storage}{Total\ Precipitation} \tag{4.8}$$

Although a simple index for environmental sustainability is considered here, other criteria that influence sustainability such as pollution control, energy consumption and maintaining the natural hydrological cycle can be evaluated in this stage. By increasing the number of the criteria and indicators, Multi-criteria decision analysis methods such as AHP/ANP and TOPSIS can be applied [117] to aid the decision-making process.

4.4 Results and discussion

4.4.1 Cost

The proposed framework has been applied to the case study for four different DCs of 100%, 66%, 33% and 0% respectively for layouts with one, four, seven and ten outlet(s). Recall from Chapter 3, DC is defined using the following equation.

$$\mathrm{DC} = 100 \times \left(1 - \frac{N_{SO} - 1}{N_{PO} - 1}\right) \quad (\%) \tag{4.9}$$

where N_{SO} is the number of selected outlets from a list of candidates, and N_{PO} is the total number of possible candidate outlets. Table 4.2 summarizes the results of this analysis and provides a comparison between HGBGIs found in the current chapter and gray designs found for the same case study in Chapter 3. Therefore, there are eight designs for four DC with and without GBIs. Figure 4.4 to 4.7 presents four designs with GBI. It is recognized that GBIs can significantly diminish the LCC of the totally centralized (DC=100%) design. The LCC of HGBGI for the design with DC=66% is roughly equal to its gray design. However, for more decentralized alternatives (DC=33% and 0%) the gray designs are 11 and 7 percent cheaper than designs with GBIs respectively. Table 4.2 reveals that the GBIs have more impact on the more centralized network of pipes. The LCC of HGBGI for the design with DC=100% is 22% cheaper than the gray only design. Moreover, the reduction in average diameter (D) and average invert depth (E) is higher in more centralized scenarios. The reason could be capturing storm-water in each sub-catchment reduces the flow in all downstream parts of it while in the more decentralized network this only has effects on the part of the pipe network that is equipped with that GBI. For the same reason, more centralized layouts tend to use a higher number of GBIs than more decentralized layouts as can be seen in Table 4.2. likewise, Figures 4.6 and 4.7 show that in the decentralized scenarios, the larger sub-systems tend to use proportionately more GBIs and follow the similar pattern that I observed in the whole system. Maximum and average pipe diameter and invert depth have been decreased in all scenarios using GBIs. Most surprisingly, in the design with DC=66%, adding GBIs reduced the maximum pipe diameter from 2 m to 0.8 m. This scenario has used more GBIs than all other scenarios.

Table 4.2: *Comparison between different scenarios*

Design	LCC[a]	Cap GBI	Cap. Gray	O&M GBI	O&M Gray	Avg. D (m)	Avg. E (m)	Max D (m)	Max E (m)
100	422700	0	138860	0	283840	0.88	2.57	2	6.78
100+GBI	329672	86833	60780	57819	124240	0.43	1.69	1.5	6.39
66	310170	0	101900	0	208270	0.68	2.01	2	5.22
66+GBI	295964	97339	43957	64819	89849	0.37	**1.57**	**0.8**	4.91
33	**250953**	0	82433	0	168520	0.64	2.13	1.5	4.72
33+GBI	276370	76841	**48737**	51171	**99621**	**0.35**	1.64	1	**4.3**
0	253197	0	83177	0	170020	0.63	2.16	1.5	4.98
0+GBI	269510	**59731**	55849	**39770**	114160	0.40	1.67	1	4.52

[a] All costs are in million Rials

Figure 4.4: *DC= 100% + GBI*

Figure 4.5: *DC= 66% + GB*

Figure 4.6: *DC= 33% + GBI*

Figure 4.7: *DC= 0% + GBI*

4.4.2 Resilience

To evaluate the resilience of each design using Equation 4.7, four design storms with the return periods of 10, 20, 25 and 50 years (6 hours duration) and total depths of 38.3, 46.7, 49.5 and 58.5 mm, respectively, are used. Figure 4.8 shows the results of this analysis. As can be interpreted from Figure 4.8, for all flood scenarios gray networks without GBIs perform much better. The best performance is provided by DC=66%, followed by DC=33% and 0% that show similar performance. The totally centralized design (DC=100%) has the lowest performance despite its larger gray storage capacity (bigger average pipe diameter) because its performance completely depends on its single main collector (the pipes with D=2m at upstream of outlet number 4). As soon as that main collector encounters its maximum capacity, all the upstream areas start to be flooded. On the other hand, the decentralized systems give diverse options for drainage, and the performance of each part does not disturb other parts as discussed in Chapter 3. As expected, GBIs do not perform as well as gray practices in facing severe rain-storms. The resilience of all HGBGIs is significantly diminished for all the flood scenarios. The more a scenario has used GBIs, the more it is vulnerable. The average reduction of resilience for all designs from DC=100% to DC=0% in sequence is 5.9%, 17.3%, 5.9% and 4.9%. Interestingly, the design with DC=66% that had the highest resilience, shows the lowest resilience when it combines with GBIs.

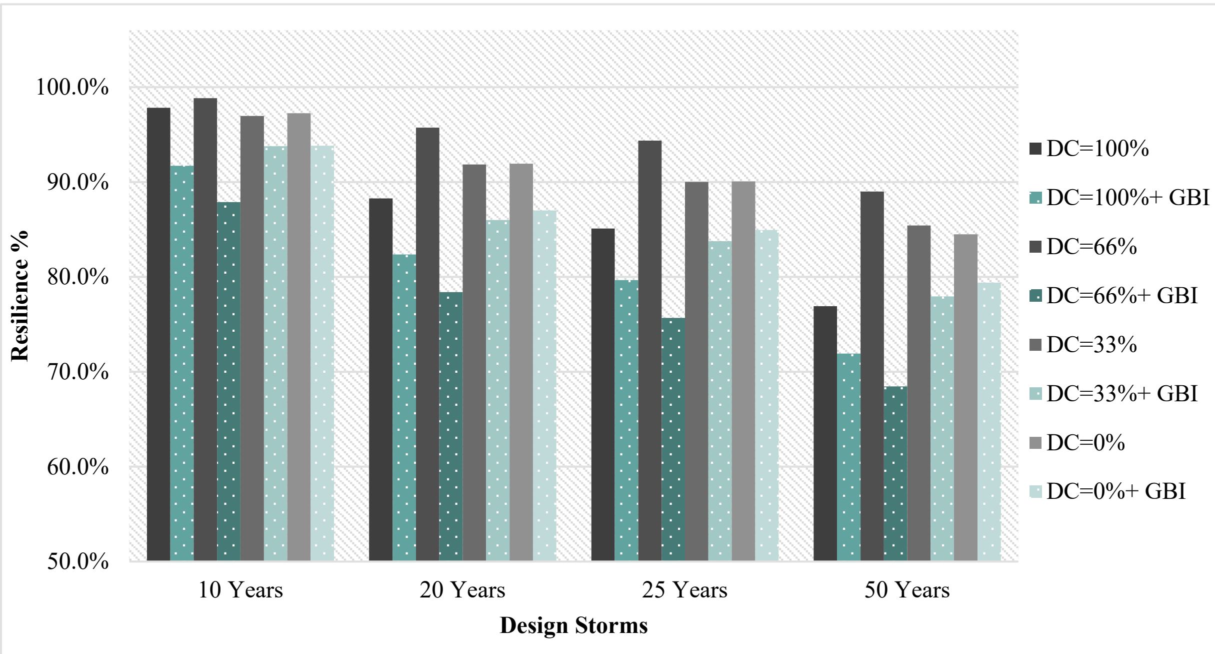


Figure 4.8: *Resilience of different design alternatives*

4.4.3 Environmental sustainability

Figure 4.9 shows the environmental sustainability of each design assessed using Equation8. As can be interpreted, the environmental sustainability of all four gray designs is the same, because pipe configuration does not affect the imperviousness in the area. The environmental sustainability is enhanced in the HGBGIs by extending the storage and infiltrating capacity provided by rain barrels and infiltration trenches. Figure 4.9 displays the flow in the outlet number 4 in all designs. This outlet is chosen because it is the only common outlet between all four design scenarios. This figure shows how decentralizing through gray and green-blue measures leads to a reduction in the peak flow. Decreasing the DC in the pipe network only reduces the amount of peak flow, but adding GBIs alters both the amount and timing of the peak flow. The maximum captured stormwater and peak flow reduction is obtained in design with DC=66%+GBI. The other two decentralized designs (DC=33&0% + GBI) also show a satisfying performance in this regard.

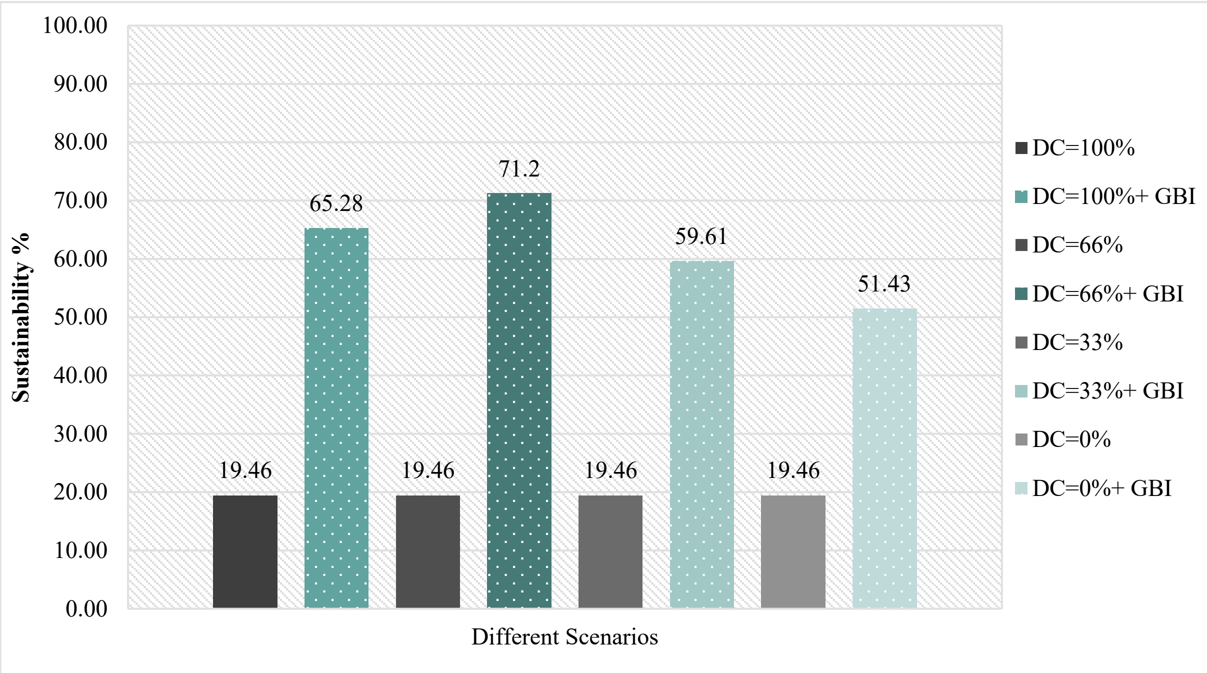


Figure 4.9: *Environmental sustainability of different design alternatives*

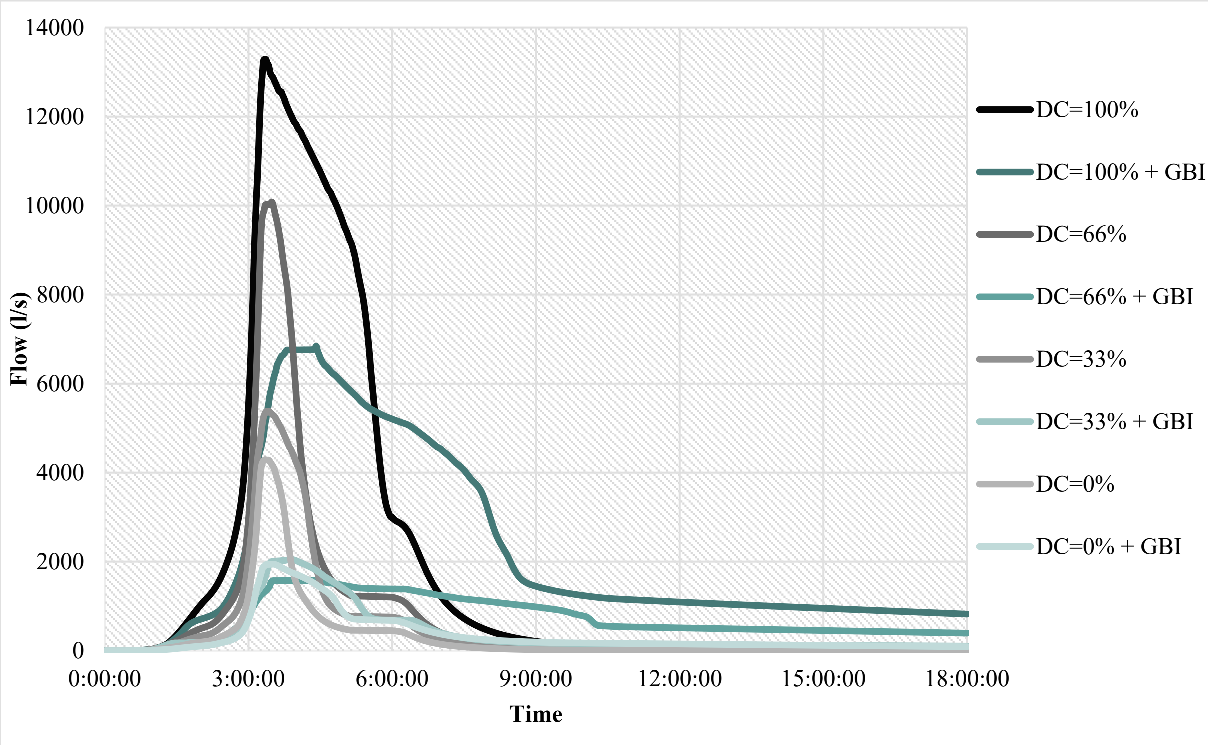


Figure 4.10: *Flow in the outlet number 4 in different design alternatives*

4.4.4 Discussion

Although the lack of capital is the main obstacle to construct a new stormwater management system, in many areas, designing these systems to be resilient and sustainable is also vital. Changing rain patterns due to climate change, rapid land-use change due to urban development and changing regulations due to public awareness and demand for prosperity are among the most important threats that force new systems to be more resilient and adaptable. Sustainability of new infrastructures is also very crucial in a country and must not anymore be sacrificed for the thirst for development that has left behind many long-term degrading environmental impacts (Madani et al., 2016). Iran and especially the Khuzestan province, where the study case is located, is currently experiencing severe water problems. Frequent droughts coupled with overabstraction of surface and groundwater are leading to drying lakes, rivers and wetlands, declining groundwater levels (Madani et al., 2016), land subsidence (Dehghani et al., 2013), deteriorating water quality, soil erosion, desertification and more frequent dust storms (Madani et al., 2016; Madani, 2014). Therefore, there is a need for a paradigm shift from a structural-based approach for development to a more sustainable and environmental friendly approach to alleviate these issues and prevent more tragic conditions in the future.

The results of this chapter show that HGBGIs of stormwater management systems can economically compete with conventional gray-only pipe networks for the test case. GBIs are more effective on more centralized networks; however, the hybrid solutions are only marginally more expensive than traditional solutions for more decentralized alternatives. They are more sustainable and environmental-friendly however, resiliency is sacrificed for this gain. Therefore, the optimal degree of centralization depends on the objectives and it differs for cost, resilience and sustainability. This fact indicates that better decisions can be made only using a multi-objective optimization framework. As an example, a tiny increase in pipe diameters might lead to significant gains in resiliency by a reasonable increase in costs and a small decrease in sustainability.

Although In this study the layout sub-problem is solved only for the cost minimization, the layout of pipes has also a significant effect on the system resilience [54]. For that reason, the next chapter focuses the combined green-gray optimization in a joint approach, and directly uses multi-objective optimization to find optimal trade-off solutions between cost, reliability, resilience and sustainability.

4.5 Conclusion

A simulation-optimization framework to optimize urban drainage systems (UDSs) considering hybrid green-blue-gray infrastructures (HGBGIs) and different degrees of centralization (DCs) has been developed and tested using a real case study, a part of the city of Ahvaz in Iran. The following conclusions can be derived from the results of the case study:

- HGBGIs can economically compete with traditional gray-only pipe networks.
- GBIs are more effective on more centralized networks for cost reduction.
- The optimal DC depends on the objectives and it differs for cost, resilience and sustainability.
- The more a UDS uses green-blue infrastructures (GBIs) to reduce the size of pipes, the more vulnerable it is for rainstorms that are more severe from the design storm.
- Using GBIs can alleviate ecological-environmental water-related problems in the area by reducing the peak flow, storing the water and recharging the groundwater surfaces.
- The results of this study are case dependent and cannot be directly transferred to another area with different specifications, however, the proposed framework can be applied for the design of any new UDS.
- More expensive GBIs like green roofs and permeable pavements can be considered in the proposed framework if the long-term benefits of them (e.g. energy reduction, pollution removal, alleviate the urban heat) are assessed.
- Optimization of new green-blue-gray UDSs should be done in a joint multi-objective framework for better decision making. This, however, will significantly increase the required computational effort. Therefore, some modifications will be needed to reduce the search space and make the problem solvable in a reasonable time.

Chapter 5. Sustainable planning of hybrid decentralized UDSs

Most of the content of this chapter has been published in the *Journal of Water Resources Planning and Management* under the title "Towards sustainable urban drainage infrastructure planning: a combined multiobjective optimization and multicriteria decision-making platform" [43]. Besides, some parts of this chapter has been published in the *Water* under the title "Integrating Structural Resilience in the Design of Urban Drainage Networks in Flat Areas Using a Simplified Multi-Objective Optimization Framework" [44].

Summary

This chapter aims to introduce a generic solution in the context of a multicriteria decision making (MCDM) platform to (1) facilitate the optimization of hybrid (de)centralized urban drainage infrastructures with many decisions and often conflicting objectives (reliability, resilience, sustainability and construction costs), (2) investigate the trade-offs between performance indicators and the system configuration, and (3) avoid conflicts between optimization analysts and decision makers by involving the latter in different stages of the planning procedure. For this purpose, first, all optimum design scenarios of hybrid UDSs are generated through multi-objective optimization (MOO). Then, a platform based on MCDM is presented to comprehensively analyze the solutions found by MOO and to rank the solutions. For the sake of demonstration, the proposed framework is applied to a real case study. The results confirm the ability of the proposed framework in handling many decisions, objectives, and indicators for solving the abovementioned complex optimization problem in a plausible time by delivering realistic solutions. In addition, the results demonstrate the important role of the degree of (de)centralization (DC) and layout configuration in obtaining the optimal solutions.

5.1 Introduction

UDSs are traditionally designed using hydraulic reliability-based approaches. These approaches assure a sufficient hydraulic capacity to convey the runoff of a specific design storm [4, 30]. For such approaches, the literature has widely addressed methods that combine mathematical simulation models with optimization/decision-making methods to design, rehabilitate, or retrofit UDSs [30]. Nevertheless, the performance of existing UDSs in various cities is negatively affected by multiple and uncertain threats, such as climate change, rapid and uncontrolled urbanization and aging infrastructure. Together, these threats cause more frequent and more severe urban flooding with adverse consequences on society, the economy and the environment [35].

Therefore, conventional design approaches have increasingly been questioned. According to various publications, UDSs should be not only (1) reliable during normal loading conditions to minimize failure (flood) frequency but also (2) resilient to extreme loading conditions to lessen the span and extent of floods, along with (3) pursuing sustainability in the long term to accomplish economic, environmental and social aims [35, 36, 118]. Additionally, conventional design approaches have increasingly been questioned because of their centralization and due to their focus on pipe networks only [7, 8]. Recent studies in urban water management favor decentralized solutions for UDSs [1, 28] to decrease life cycle costs [41, 119], to increase system resilience, flexibility and adaptability [118–120] and to reduce adverse impacts on the environment [11, 12].

To achieve decentralization, the application of green-blue infrastructures (GBIs), such as green roofs, permeable pavements, infiltration trenches and rain barrels, as alternatives is receiving increasing attention [96, 117].

GBIs are flexible and adaptable measures that provide several cobenefits in addition to flood risk reduction. The cobenefits include water quality improvement, recharging groundwater, water harvesting, restoring the hydrologic characteristics of the site, increasing urban amenities and alleviating the urban heat island effect that are aligned with environmental and social aims of sustainability [19, 21, 31]. However, GBIs are a relatively expensive investment and have poor resilience to extreme loading conditions [27, 31].

In addition to using GBIs, decentralization can be obtained using conventional gray infrastructures (CGIs) by dividing heavily centralized pipe networks into several parts with multiple outlets or by employing distributed storage tanks [7, 41]. In contrast to GBIs, pure CGIs are proven to have a degrading impact on the environment due to the discharge of polluted stormwater or wastewater to bodies of water. They are also challenging to upgrade and expensive to maintain (not adaptable) [12]. However, in comparison with GBIs, GCIs need less capital investment and show higher functional resilience to cope with intense rainstorms [27, 31].

To combine the advantages of GBIs and CGIs, hybridization to so-called hybrid green-blue-gray infrastructures (HGBGIs) has been evaluated in recent studies of urban water management [39]. The results showed that HGBGIs tend to complement each other [20]. Hence, HGBGIs might be the most promising approach to handle the challenges mentioned above in the modern urban management era to achieve higher reliability, resilience and sustainability at a lower price [28, 29, 31, 39, 121].

To conclude, future UDSs need to be reliable, resilient and sustainable. HGBGIs seem to be the most supportive approach to achieve these goals. Notwithstanding, considering all combinations of possible alternatives among CGIs and GBIs and considering the many possible objectives, designing new UDSs constructs a notably complex optimization problem. To jointly optimize such complex systems, obtaining the optimum layout of the pipe network considering the different degrees of decentralization (DCs), sizing the sewers and selecting the type, size and location of GBIs are the subproblems that need to be decided simultaneously. Each of these optimization subproblems contains many decisions and technical and hydraulic constraints. In addition, there are different objectives that, in some cases, conflict with each other [38] and increase the dimension of the problem complexity. As an example, increasing the size of pipes in the network might increase reliability and functional resilience but is not necessarily financially viable and environmentally sustainable.

Currently, mathematical optimization is a promising approach in the field of urban water management to aid in finding a preferred option from feasible solutions, such as a range of designs, planning, operations, management and policy scenarios [122, 123]. Environmental models, such as urban drainage models (e.g., SWMM), are utilized broadly to aid these decision-making procedures by assessing the performance of different alternatives [65]. My review of existing methodologies and frameworks for MOOs of UDSs shows that the so-called reliability-based approach is the most common approach to deal with this problem. In this approach, the performance of either gray alternatives or green-blue scenarios is evaluated and optimized for a single design storm or a limited number of separate design scenarios [5, 19, 21, 27, 29, 53, 100, 101, 105, 108, 111, 124, 125].

A limited number of works in the literature have incorporated resilience in their optimization formulation by including functional resilience [14] or structural resilience [54].

Many studies have utilized multicriteria decision analysis (MCDA) instead of optimization in their proposed frameworks to consider various characteristics of resilience and different aspects of sustainability [119]. MCDA approaches empower decision makers to cover a full range of decision-relevant indicators [117] in a plausible run-time but only for a limited number of predefined scenarios.

In addition, there have been a few studies that combine optimization and MCDA techniques. As an example, Sweetapple et al. (2017) [126] proposed a general framework for reliable, robust, and resilient system design. This framework contained three key components: (1) an MOO was applied to design the system under standard loading, and a set of Pareto-optimal solutions were obtained, (2) the solutions on the Pareto fronts that provide an acceptable level of robustness were subsequently used for resilience analysis, and (3) the solutions that reached this step (were reliable and resilient) were ranked based on their performance objectives and the priorities of the decision makers. Although this is a very promising approach to include robustness and resilience in system design, the fundamental optimization step is based on reliability only. Therefore, resilience and robustness analysis is performed for only a limited number of solutions that cannot guarantee a global optimum.

To conclude, my review revealed that, at the moment, no UDS optimization tool or framework exists in the literature that can simultaneously consider various performance indicators, many decisions and different technical and practical constraints in its structure. The available literature has solved this problem by either isolating the objectives or the alternatives. Therefore, the attributes and relationships between these operational and strategic system objectives (reliability, functional and structural resilience and sustainability) and UDS configuration (layout and degree of (de)centralization of CGIs) are still unclear [36, 39].

In addition to the challenges mentioned above, the acceptance of the solutions obtained from mathematical optimization by system authority institutions (decision makers) can be challenging [127]. Decision makers or stakeholders have valuable experience from several years of working and confronting real-world challenges that might be ignored by optimization analysts. Additionally, there are other engineering and practical considerations and desires that cannot be formulated in the mathematical optimization procedure. Some of these desires imposed by decision makers might even be irrational. However, if decision makers as clients feel undervalued and unheard or the presentation of the model results and optimization procedure are not transparent to them, the buy in to the optimization results might be decreased [127, 128]. Therefore, to develop trusted strategies that are likely to be adopted in practice, decision-maker engagement should be encouraged in all phases of the optimization frameworks applied to water resource problems [122, 127, 129, 130].

This chapter aims to address the challenges mentioned above and fill the identified gaps in the literature. The key contributions of this chapter are the following:

1. To introduce a combined multiobjective optimization (MOO) and multicriteria decision-making (MCDM) platform that aids the sustainable planning of modern hybrid (de)centralized urban drainage infrastructures.
2. To provide the tools and materials to explore the trade-offs between operational and strategic system indicators (e.g., reliability, resilience and sustainability) and system configuration (network layout and degree of (de)centralization).
3. Finally, to enable urban drainage operators and water authorities to participate in decision making.

The remainder of this chapter is structured as follows: in the next section, first, the definitions of reliability, resilience, and sustainability indicators in the field of urban drainage are given; then, the proposed framework is presented in detail. The proposed framework is demonstrated and discussed by solving a case study. The last section concludes the chapter and provides recommendations for further investigations.

5.2 Reliability, resilience and sustainability in UDSs

Hashimoto et al., (1982) [131] proposed three criteria for assessing the possible performance of water resource systems to support the evaluation and selection of alternative designs and operating policies of water resource projects. From their definition, reliability describes how likely a system is to fail, resiliency measures how quickly it recovers from the failure, and vulnerability estimates how severe the consequence of failure might be. To reduce the computational burden, especially when complex system-response models are used, Maier et al., (2001) [132] introduced an efficient approach based on the first-order reliability method for computing reliability, vulnerability, and resilience.

Later, Butler et al. (2014) [35] introduced the ‘‘ Safe & SuRe’’ framework for water management. Based on the ‘‘Safe & SuRe’’ approach, reliability is the bedrock of resilience and sustainability. These authors defined reliability as the degree to which a system minimizes the frequency of failure over its design life when subject to standard loadings. In the context of urban drainage, service failure means failure to comply with the levels required by regulations, e.g., when sewer flooding or combined sewer overflows (CSOs) violate a given threshold [36]. Therefore, current approaches for hydraulic reliability-based design, retrofitting and rehabilitation concentrate on avoiding hydraulic failures occurring from a design storm of a given return period [4].

Binesh et al. (2019) [133] formulated three types of reliability for describing different angles of UDS performance: (1) occurrence reliability, (2) temporal reliability and (3) volumetric reliability. Occurrence reliability is defined by the number of times a satisfactory state (e.g., no surcharging in the system) has occurred during a certain number of time steps. Temporal reliability represents the amount of time the system remains in the satisfactory state divided by the total range of time considered. Volumetric reliability considers the ratio of water volume conveyed safely through the drainage system to the total runoff volume generated from rainfall [133].

The above definitions of reliability do not consider other sources of failure that the system is not designed for [134], such as structural failure (e.g., pipe blockage, climate change and urbanization). Here is where the concept of resilience can help. According to [35], resiliency describes the response of the system after failure to unforeseen loading conditions. They define resiliency as “the degree to which the system minimizes the level of service failure (magnitude and duration) over its design life when subject to exceptional conditions.” For UDSs, two types of failure may occur, namely, functional and structural failure. The difference is in the type of threats that endanger the system. Functional failure is induced by threats that alter the load in the system, such as climate change, urbanization and extra infiltration, while structural failure is triggered by threats that lead to faults of single or multiple components in the system, such as pipe blockages, pump failure and clogging in infiltration trenches [30, 134]. Therefore, it is crucial to consider different threats and their combinations to build resilience in UDSs. Increasing the resilience of UDSs can be obtained by enhancing two properties of the systems: redundancy and flexibility [30, 134].

Redundancy is defined as the degree of overlapping functionalities in a system. It permits the system to continue vital functions while formerlys redundant elements break or take on new functions [135]. Flexibility, on the other hand, is defined as the system’s intrinsic ability to be modifies or reconfigured to preserve adequate performance levels when subject to multiple (varying) loading conditions [118]. In UDSs, redundancy is enriched by presenting multiple elements providing similar functions, such as additional storage tanks and parallel pipes, and increasing spare capacity in critical points of the network. Flexibility can also be achieved in

UDSs by designing future-proof options. These include the use of distributed (decentralized) elements, such as GBIs [37].

Considering resilience in the design process of UDSs exponentially increases the complexity of the design problem. Different threats and their combinations need to be taken into account. Additionally, trying to enhance inbuilt system flexibility and redundancy introduces extra decision variables into the optimization problem.

Notwithstanding, the complexity added to the problem by introducing resilience is still not enough for future UDSs. Sustainability is another concern that needs to be addressed in the framework for designing future UDSs. Sustainability, in general, ought to simultaneously address today's demands and the impacts on future generations. It requires a holistic view that considers environmental, economic and social aspects equally [38]. Butler et al. (2014) [35] define sustainability as "the degree to which the system maintains levels of service in the long-term while maximizing social, economic, and environmental goals." This definition can bring new objectives, such as pollution control, rainwater usage, public acceptance and annual energy savings, to the problem. These additional objectives cannot be accounted for in reliability and resilience indicators [36].

Reliability, resilience and sustainability are three aspects of UDSs that should be pursued simultaneously during the decision-making procedure. Usually, better decisions lead to an enhancement of all aspects, but in some cases, the same decision could lead to an improvement in one aspect(s) and a worsening of other(s), or vice versa [38]. For example, increasing pipe diameters in the system could improve system reliability, resiliency, and public sustainability. However, it can at the same time decrease environmental and economic sustainability by conveying more pollution to water bodies and requiring more capital investment. Alternatively, adding parallel pipes does not necessarily enhance the system reliability; however, it improves system resilience and public sustainability.

Butler et al. (2014) [35] introduced a pyramidal structure to explain the connection between reliability, resilience and sustainability. This pyramid demonstrates that resilience should build upon reliability, and sustainability should build upon resilience. Although these indicators are inevitably interlinked, the complicated relationship between them is still unknown. Therefore, one aim of this chapter is to investigate the trade-offs between life cycle costs, DC and performance indicators (e.g., reliability, resilience and sustainability).

5.3 Proposed framework

The proposed framework for sustainable hybrid (de)centralized urban drainage infrastructure planning is shown schematically in Figure *5.1*. This framework consists of three main steps: (1) system definition, (2) simulation optimization and (3) final decision making. The details of all steps, subprocesses and algorithms are given in the following sections.

5.3.1 Step 1: System definition

First, stakeholders or decision makers introduce all types of urban drainage systems and technologies that they wish to be considered in the design process in addition to all technical and practical considerations. Any screening tool for GBI type or sanitation technology selection, location identification and narrowing of the dimensioning variables without detailed optimization can be employed in this step. Such primary screening restricts the search space and enhances optimization efficiency [20]. Some examples of such screening tools can be found in [136–142].

Then, all combinations of the proposed systems and technologies must be generated systematically to perform the mathematical optimization. To do that, in *step 1a*, a base graph for the pipe network is outlined. This base graph includes all drainage feasibilities concerning the street alignments, topology, barriers, watercourses, and existing sewers in the area under design. In *steps 1b* and *1c*, the candidate locations of the outlets, the potential type, size and location of GBIs, and if necessary, treatment facilities, such as on-site treatment solutions, constructed wetlands or traditional wastewater treatment plants are determined. These steps provide the potential decision variables $\boldsymbol{d}$ of the optimization problem (Eqs. 5.1 and 5.2). Next, in *step 1d*, all relevant performance indicators, such as reliability, resilience and sustainability, for MOOs need to be defined. To prepare for the simulation-optimization step (step 2), it is crucial to define simple performance indicators that are inexpensive to evaluate. Otherwise, the optimization in step 2 might take a long time to converge and get close to acceptable solutions. For example, design storm(s) (different types of design storms can also be considered for a more robust design) or multiple extreme events derived from historical rain data should be used instead of recorded time series when evaluating hydraulic reliability or functional resilience. The discussion and references provided in the previous section can help readers select appropriate indicators for a specific problem.

Finally, in *step 1e*, a numerical UDS model (EPA SWMM in this study) is constructed to evaluate the predefined performance indicators. In this step, the model parameters (e.g., impervious area, manning roughness, and soil characteristics) are calibrated or estimated. Design conditions and physical and technical constraints (e.g., design loadings, maximum velocity, minimum slope, maximum buried depth, and water quality standards) are defined. Then, based on the data mentioned above, a base model is constructed. From the base model, different design schemes will be generated during the simulation-optimization step (step 2) by decoding the design variables, as explained in the next section.

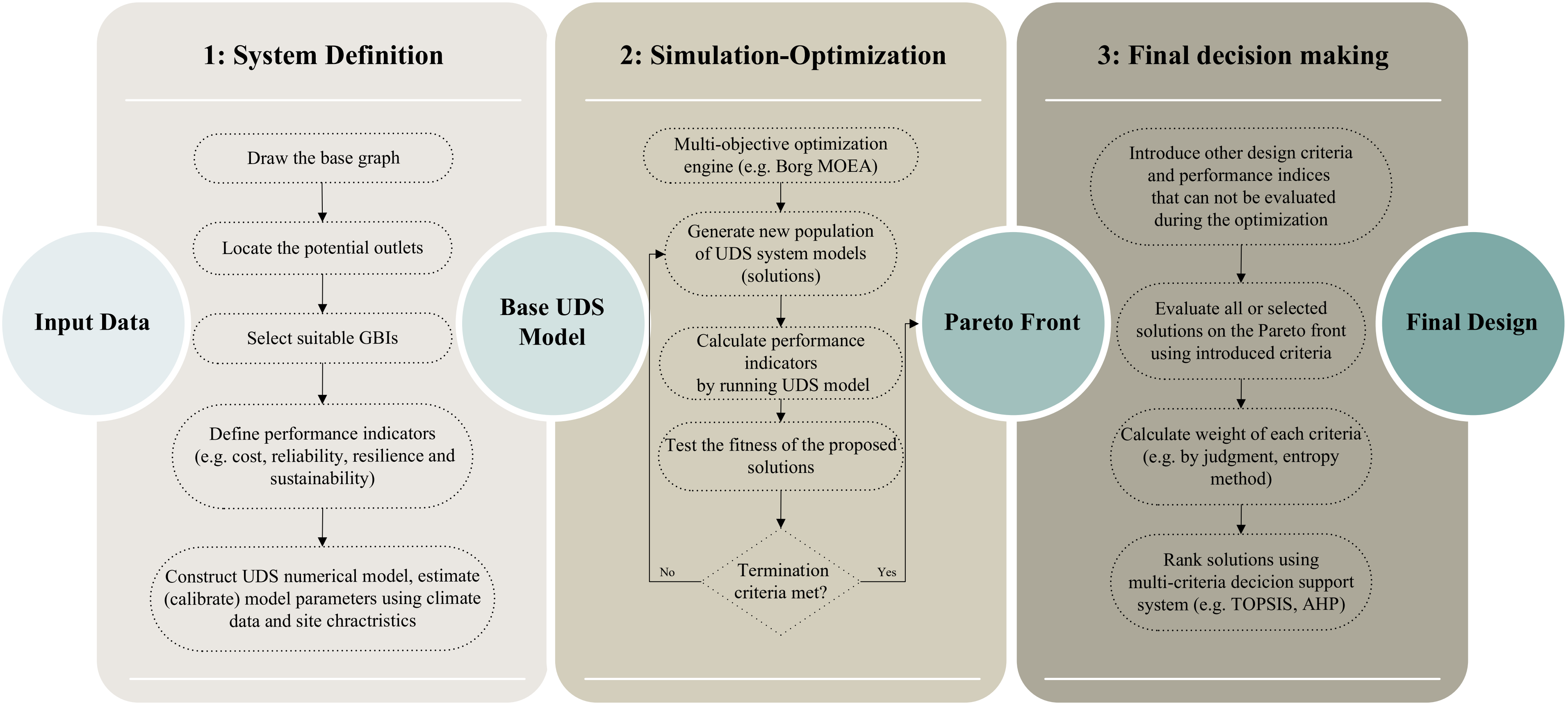


Figure 5.1*: The proposed framework*

5.3.2 Step 2: Simulation-Optimization

Mathematically, the multi-objective optimization of HGBGIs can be formulated as:

$$\boldsymbol{d}_{\mathbf{opt}} = \arg \max_{\boldsymbol{d} \in \boldsymbol{D}} [\langle f_{Indicator\ 1} \rangle, \langle f_{\text{Indicator 2}} \rangle, \dots, \langle f_{\text{Indicator}\ n} \rangle] \tag{5.1}$$

$$\boldsymbol{d} = [\langle DC \text{ and } layout\ parameters \rangle, \langle CGI\ parameters \rangle, \langle GBI\ parameters \rangle] \tag{5.2}$$

where $\boldsymbol{d}_{\text{opt}}$ is the optimal choice for the decision variables $\boldsymbol{d}$ that define the UDS. Decision variables $\boldsymbol{d}$ contain at least elements of three sub-problems (Equation 5.2):

1. DC and $layout\ parameters$ that determines the connectivity between gray components of the system in each part and the distribution of the system as a whole when multiple outlet candidates are available. Here DC is defined using Equation 5.3 introduced in Chapter 3.

$$\text{DC} = 100 \times \left(1 - \frac{N_{SO} - 1}{N_{PO} - 1}\right) \quad (\%) \tag{5.3}$$

where, N_{SO} is the number of selected outlets from a list of candidates, and N_{PO} is the number of possible candidate outlets.

2. $CGI\ parameters$ that specify the size of each conventional gray component such as pipe diameters, slopes, location and technical details of pump stations, as well as the location and size of storage tanks.
3. $GBI\ parameters$ that indicate the type, size and location of GBI.

$\boldsymbol{D}$ is the feasible space where all structural, technical, hydraulic, environmental, and economic constraints are met. $f_{\text{Indicator i}}$ is the i^{th} performance indicator or objective function.

To do a systematic optimization, it is vital to systematically generate various HGBGI schemes that satisfy all technical and physical constraints. To do this, three different modules are employed within the proposed framework; (1) the *hanging gardens algorithm* to generate feasible layouts with an arbitrary degree of (de-)centralization, (2) an adaptive algorithm to hydraulically design the generated layouts and (3) an algorithm to define the type, size and location of GBIs. Figure 5.2 schematically shows the simulation-optimization procedure proposed in this study and demonstrates the connection between different algorithms.

The *hanging gardens algorithm*, introduced in chapter 3 [41], needs $2 \times (NL + N_{PO})$ decision variables to generate one feasible layout. NL is the number of loops in the base graph and N_{PO} is the number of possible outlets.

For each generated layout, CGI specifications such as pipe diameters, pump stations and invert elevation are designed in a way that satisfies all hydraulic and technical constraints. To satisfy technical constraints like the telescopic pattern, minimum cover depth, maximum excavation depth, and minimum and maximum slope, the adaptive approach introduced in [52, 55] is used. The hydraulic constraints of maximum velocity and no flooding are handled using a penalty function during optimization. If NP is the number of pipes in the UDS, this algorithm needs $3NP$ decision variables to design the system hydraulically; one per pipe to assign the size of a pipe, one to assign the slope of it, and one that determines whether there is a pump station upstream of a pipe. Finally, distributed measures, here GBIs, are added to the designed UDS using an adaptive algorithm to construct a hybrid green-blue-gray UDS, as explained in the following paragraph.

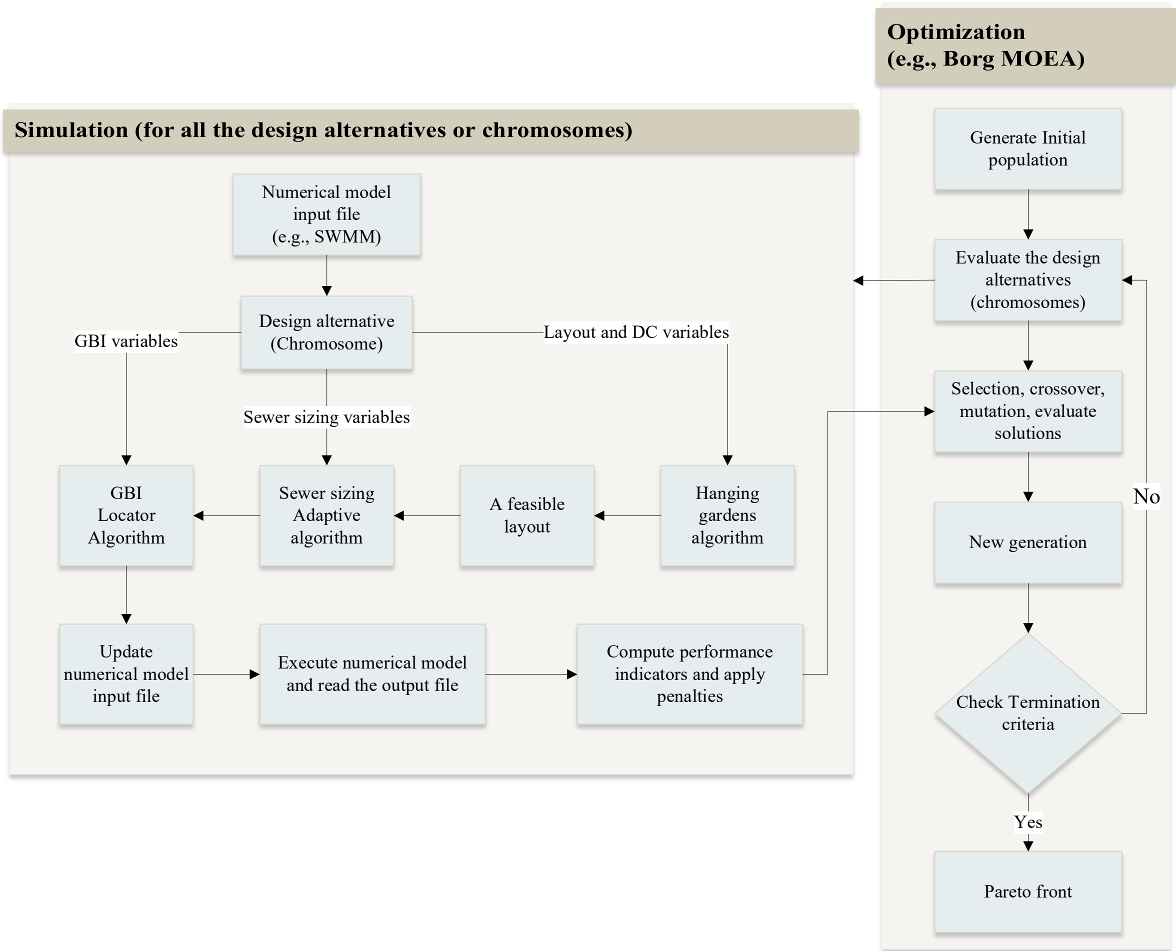


Figure 5.2: *Simulation-optimization procedure*

To generate a hybrid alternative, three extra decision variables for each sub-catchment (in total $3NS$ decisions, where NS is the number of sub-catchments) is needed. For each sub-catchment, a binary variable decides whether that sub-catchment is equipped with any GBI or not. The second variable determines the type of GBI(s) from a list of feasible GBI candidates for each sub-catchment using Eq. 5.4. The third variable defines the size of GBI(s) considering the minimum and maximum feasible size of each measure using Eq. 5.5.

$$GBI\_Type_i = round(1 + (NGBI_i - 1) \times gbi\_type_i) \tag{5.4}$$

In which gbi_type_i is a uniform random variable generated by the optimization engine, $NGBI_i$ is the number of GBI(s) candidates in sub-catchment i, and GBI_Type_i is the decoded type of GBI(s) in that sub-catchment. As an example, suppose the candidate list of GBI(s) in one sub-catchment is as follows, and gbi_i randomly generated by the optimization engine is 0.67.

$$GBI\_Candidate_i = \left\{ \begin{array}{c} 1\text{: } Rain\ barrel \\ 2\text{: } Infiltration\ trench \\ 3\text{: } Green\ roof \\ 4\text{: } Rain\ barrel + Infiltration\ trench \\ 5\text{: } Rain\ barrel + Green\ roof \\ 6\text{: } Infiltration\ trench + Green\ roof \\ 7\text{: } Rain\ barrel + infiltration\ trench + Green\ roof \end{array} \right\}$$

Using equation 15, $GBI_i = round(1 + (7 - 1) \times 0.67) = 5$, which means that $Rain\ barrel + Green\ roof$ is selected for that catchment.

GBI_Size_i is handled as follow:

$$GBI\_Size_i = GBI\_Size_{min,i} + (GBI\_Size_{max,i} - GBI\_Size_{min,i}) \times gbi\_size_i \tag{5.5}$$

In which gbi_size_i is a uniform random number generated by the optimization engine, $GBI\_Size_{max,i}$ and $GBI\_Size_{min,i}$ are, respectively, the minimum and maximum permissible size of GBI_Type_i, and GBI_Size_i is the decoded size of GBI_Type_i in that sub-catchment.

Using the abovementioned algorithms, any arbitrary generated set of decision variables $\boldsymbol{d}$ constructs a feasible HGBGI scheme. These algorithms together with performance indicators and a numerical model for evaluating the system performance (calculating values of indicators) provide essential tools and materials to do the MOO. The MOO engine generates a set of Pareto-optimal solutions using different sets of decision variables that satisfy the defined constraints. There are several genetic algorithms (GAs) MOO engines (e.g., NSGA-II, BORG, GALAXY) in the literature that can be employed for the current optimization purpose (BORG in our study).

5.3.3 Step 3: Final decision making

The MOO in the previous step identifies the solutions that dominate other solutions and constructs the Pareto front. As there is no clear preference between the solutions on the Pareto front according to the optimization objectives from step 2, a final decision step is required to help decision makers select an appropriate option from the Pareto front. This step can account for aspects that could not be covered by the MOO in step 2.

First, the MOO compares different solutions by only evaluating a limited number of objective functions. However, some criteria cannot be considered due to practical reasons such as limitations in computational power. For example, applying the global resilience analysis approach introduced by [118] to evaluate the structural resilience of UDSs requires generating and evaluating many pipe failure scenarios for each solution. Such a procedure increases the computational burden exponentially. Second (and again due to the computational burden), modeling flooding of specific locations in the urban area has to be done separately using a 2D model based on an accurate detailed DTM. This is only feasible for a few selected solutions. Third, there are other aspects that cannot be represented mathematically and criteria that cannot be quantified adequately (e.g., risk of human casualties or public acceptance).

Multicriteria decision analysis (MCDA) techniques (e.g., AHP/ANP, TOPSIS) provide the opportunity to include numerous ranges of indicators and thoroughly analyze the limited number of solutions that exceed this stage [117, 143]. Hence, in this step of the proposed framework, a full range of desired indicators is first defined. Then, the performance of all the selected solutions is evaluated. Finally, the solutions are ranked employing an MCDA technique. No specific MCDA technique is prescribed here for this purpose, as this must be suitable for the problem. For our application, in the next step, we will use TOPSIS.

The computation steps to calculate the TOPSIS scores and rank the alternatives are given in Chapter 2. As the determination of a specific weight for each index in TOPSIS is usually subjective, the entropy method was utilized to calculate the weights and reduce the subjectivity as presented in Chapter 2.

5.4 Case Study

This section intends to demonstrate the performance of the proposed framework step-by-step by applying it in a realistic case study. The case study features a section of the city of Ahvaz in Iran as previously introduced in Chapters 3 and 4.

Step 1: System Definition

<u>Steps 1a to 1c:</u> *Drawing the base graph, locating the candidate outlets, and selecting GBIs*

The case study has an area of approximately 500 hectares that is divided into 181 subcatchments (loops in the base graph), including 530 pipes (approximately 75 km length) and ten candidate outlets. In this study, we select rain barrels and infiltration trenches as GBI options. The reason for this choice, the maximum size of each measure and their suitable location in the area under design are discussed in chapter 4 [42]. After rainfall, runoff from the roofs is diverted into rain barrels to supply water for toilet flushing and household irrigation. A percentage of the runoff from impervious areas, such as roads and parking lots, and roof runoff overflowing the rain barrels are diverted into infiltration trenches. It is assumed that each apartment can be equipped with a 2 m^3 rain barrel that is available on the local market. The infiltration trenches are installed along streetscapes and can cover, on average, up to 5% of the impervious area in each subcatchment. Each infiltration trench unit is assumed to have a width of 2 m, a length of 5 m and a berm height of 250 mm. Other design parameters are assigned or estimated according to the literature as follows: vegetation volume fraction 0.1, storage (gravel) layer thickness of 1500 mm, void ratio of 0.75, seepage rate of 0.56 mm/h, drain flow exponent of 0.5 and offset height of 100 mm (Cano and Barkdoll, 2017; Chui et al., 2016; Eckart, 2015). These specifications remove the selection of GBI type and size from the optimization, and only the locations for the GBIs remain to be determined.

<u>Step 1d:</u> *Defining performance indicators*

To prepare for the simulation-optimization step (second step), it is crucial to define simple performance indicators that are inexpensive to evaluate. For this reason and to satisfy the technical criteria given in the regional guidance manual, we will use the following indicators for the case study:

Reliability: According to the local manuals, stormwater collection systems must be designed for 2- to 5-year design storms (normal loading conditions) in urban areas. The design storms can be found in the Supplemental Data. No system flooding is allowable for the selected design storm. We quantify the reliability of the system as the following:

$$Rel = \begin{cases} 0 \ if \ HPI_2 < 1 \\ HPI_5 \ if \ HPI_2 = 1 \end{cases} \tag{5.6}$$

$$HPI_T = 1 - \frac{V_{flooding}}{V_{runoff}} \tag{5.7}$$

Where, HPI_T is the hydraulic performance index of a design storm with return period T, $V_{flooding}$ is the total water that overflows the nodes, and V_{runoff} is the total runoff volume. For calculating the reliability index Rel according to Eq. 5.6, each design alternative must be evaluated one or two times, for 2- and 5-years design storms. The reliability is zero if there is any flooding in the system for $T = 2 \ years$. If the system handles the 2-year design storm properly, then its reliability is calculated using a 5-year design storm. All solutions with reliability higher than zero are acceptable; however, they might have different functional properties and construction costs.

Resilience: The case study considers both functional and structural resilience. Functional resilience accounts for magnitude and duration of failure when extreme loading conditions occur. Here a 25-years design storm is used as an extreme loading condition. Eq. 5.8, introduced by [118], is adopted to calculate functional resilience:

$$Res_{Functional} = 1 - \frac{V_{flooding}}{V_{runoff}} \times \frac{T_{flooding}}{T_{Simulation}} \tag{5.8}$$

In which Res_{fun} is functional resilience, $V_{flooding}$ is the total water that overflows the nodes, and V_{runoff} is the total runoff volume, $T_{flooding}$ is the spatial average flood duration computed for all flooded nodes in the system, and $T_{Simulation}$ is the total simulation time.

To compute structural resilience, a simple index that uses the adjacency matrix of the sewer layouts is adopted. The main idea of this index is that, when the area affected by a pipe failure is low, the structural resilience of the sewer network would be high [54]. On this basis, the structural resilience caused by every individual link (pipe) is defined as:

$$Res_{structrual,i} = 100(1 - \frac{A_i}{A_T})\ (\%) \tag{5.9}$$

Where A_i is the area connected to $pipe_i$, and A_T is the total area. To obtain a structural resilience index for the entire layout, I take the average over all pipes as follows.

$$Res_{structrual} = \frac{\sum_{i=0}^{NP} Res_{structrual,i}}{NP}\ (\%) \tag{5.10}$$

$Res_{structrual}$ can be very insensitive to large networks as there are a large number of upstream pipes with low discharges and high resilience. This restricts Eq. 5.10 from comparing alternative layouts. To reduce this effect, for large sewer networks, using only sewers with a resilience index less than the threshold 90% was suggested [54]. Therefore, $Res_{structrual,i}$ became $Res_{str,i<90\%}$. This removed the upstream branches from consideration. As a result, this caused the layout optimization to be more sensitive to designing a layout with one main collector, which conveys high sewage discharges. To account for the effect of DC on the resilience of the layout, Eq. 5.12 is developed to quantify the structural resilience.

$$SRI = \begin{cases} \dfrac{NP_{Res_{str,i>90\%}}}{NP}\left(\dfrac{\sum_{i=1}^{NP_{Res_{str,i<90\%}}} Res_{str,i<90\%}}{NP_{Res_{str,i<90\%}}}\right)(\%) \\ 100\%\ if\ NP_{Res_{str,i<90\%}} = 0 \end{cases} \tag{5.11}$$

in which $NP_{Res_{str,i>90\%}}$ is the number of pipes with structural resiliency more than 90% and $NP_{Res_{str,i<90\%}}$ is the number of pipes with structural resiliency less than 90%. SRI is equal to zero when all sewers are connected to more than 10% percent of the total area ($NP_{Res_{str,i>90\%}} = 0$) and 100% if each outfall is connected to up to 10% percent of the total area ($NP_{Res_{str,i<90\%}} = 0$).

Sustainability: I consider here environmental and economic sustainability. As the economic indicator of sustainability, I use life cycle costs (LCC) as discussed in chapters 3 and 4 [41, 42].

As a simple index for environmental sustainability, I use the ratio between the storage quantity and precipitation under storm design:

$$Sus_{Environmrntal} = \frac{infiltration\ volume +\ final\ GBI\ storage}{Total\ Precipitation} \tag{5.12}$$

Step 1e: *Constructing the simulation model*

As discussed in chapter 3 and 4, the EPA's SWMM version 5.1 software is used (Rossman, 2010) for the hydrologic-hydraulic simulation of pipe network and GBIs. The dynamic wave method is selected as the routing method because of its ability to account for channel storage, backwater effects, flow reversal and pressurized flow (Rossman, 2010). The main parameters for each sub-catchment, e.g. area, impervious area, width, slope, infiltration parameters, Manning's roughness, are estimated using Google Earth and engineering judgment.

Step 2: Simulation-optimization

I considered two simulation-optimization scenarios for the case study. In the first scenario, the proposed framework is applied to the four fixed layouts with different DC founded in chapter 3. Therefore, the problem is simplified to the optimal sizing of the sewers and the location of the GBIs. In the second scenario, layout and DC variables are also considered. This scenario provides the material to explore the effect of layout configuration on the different performance criteria.

In both scenarios, minimizing LCC is used as an optimization objective for two reasons: (1) LCC is the most determinative parameter for the stakeholders in the area, and (2) the LCC is different from other performance criteria and so present an independent problem dimension. The second optimization objective is maximizing total sustainability (Sus_{Total}). Here, I defined total sustainability as the geometric mean of reliability, resilience and environmental sustainability. The reasons to do that are: (1) considering each performance indicator as a separate objective function increases the computational effort exponentially due to the large scale of the test case (more than 1000 decision variables), and (2) as discussed earlier, these performance indicators have a pyramidal structure with an unknown relationship that can be obtained through the proposed formulation as will be explained in the following paragraphs.

$$Sus_{Total} = \sqrt[3]{Rel \times Res \times Sus_{Environmental}} \tag{5.13}$$

$$Res = \sqrt[2]{Res_{Functional} \times Res_{Structural}} \tag{5.14}$$

As mentioned before, sustainability has three aspects; public, economic and environmental. The economic aspect of sustainability is regarded here as a separate objective function (LCC). Besides, increasing reliability and resilience in Eq. 5.13 results in decreasing urban flood probability that automatically enhances public sustainability. Therefore, I can guarantee that all aspects of sustainability are acknowledged in our proposed MOO formulation. Here, for convenience of interpretation, all introduced indicators are a real number between zero and one; zero indicates the lowest performance of each indicator and one the highest performance. Generally, the geometric mean is used for considering several criteria that cannot compensate for each other, i.e. one aspect being rated as “zero” sets the overall performance to zero.

By using the geometric mean, total sustainability is zero when one or more of the indicators are equal to zero, and it is one if and only if all the indicators are equal to one. Therefore, the suggested pyramidal structure of the performance indicators can be inevitably obtained. The remaining technical and social aspects and indices will be treated in the decision making step. It is worth mentioning that the choice of performance indicators and MOO problem formulation depend on the specific problem at hand. I cannot prescribe any general formulation here. However, any MOO formulation can be handled using our proposed framework.

For the second scenario, maximizing DC is also used as an extra objective function. This objective means that, for a certain amount of LCC and total sustainability, solutions with higher DC (lower number of parts or outlets) are selected. Doing this allows the optimization engine to explore the feasible space fully and determine the influence of DC on the performance indices. Therefore, the general optimization problem for the test case is reformulated and simplified as follow:

$$\boldsymbol{d}_{\text{opt}} = \arg \max_{\boldsymbol{d} \in \boldsymbol{D}} [\langle -\text{LCC} \rangle, \langle -\text{DC} \rangle, \langle Sus_{Total} \rangle] \quad (5.15)$$

$$\boldsymbol{d} = [\langle DC \text{ and } layout\ parameters \rangle, \langle pipes\ diammeter \rangle, \langle GBIs\ location \rangle] \quad (5.16)$$

Table 5.1 summarizes the number, type, and function of decision variables vector $\boldsymbol{d}$ for the test case.

Table 5.1: *Number, function and type of decision variables and the algorithms that use them*

Algorithm	Number of decision variables	Function	Type
Hanging gardens algorithm	= 2 (NL+N_{PO}) = 2 (181+10) = 382	To determine the layout configuration and DC	Binary & Real
Sewer sizing adaptive algorithm	= NP = 530	To determine the size of pipes	Integer
GBI locator algorithm	= NS = 530	To determine the location of the GBIs	Binary
	Total = 1093		

In this chapter, the Borg Multi-Objective Evolutionary Algorithm [68] is employed for the MOO based on its successful application for water resources problems [98]. More detail about this algorithm is given in Chapter 2.

Step 3: Final decision making

As described earlier, some solutions from the Pareto front obtained in the previous step are selected in this step for more comprehensive assessments and then for the final decision. The SWMM simulations for the selected solutions are executed to calculate all additional indicators and rank the solutions. Besides the simple indicators used in the simulation-optimization step, 15 additional indicators (Table 5.2) are used here for final decision making. These indicators are evaluated under continuous simulation. The continuous simulation uses six-months of rainfall data from October 2018 to March 2019. This period is recognized for its extreme events in recent years that caused a lot of trouble in the area, such as flooded streets, infectious diseases, traffic jams and triggered public protests. Total rainfall depth during this period is 268.4 mm, which is 45% more than the 30-years average precipitation. The maximum precipitation in 24 hours during this period is 45.1 mm. The proposed indicators are divided into four different categories:

(1) **Technical and construction concerns (structural resilience):** This group of indicators deals with issues regarding the construction and operation of UDS. The pipes' diameter and buried

depth are primary concerns. Installing pipes larger than 1m is problematic in the already existing narrow streets in our case study. Moreover, as the average groundwater level in the area under design is 2.5 m below the ground surface, installing pipes deeper than this depth is costly and increases the risk of inflow. In extreme flood conditions, such additional loads can cause pipe blockage, reducing the structural resiliency.

(2) Environmental sustainability: I assessed the environmental sustainability of each scenario, measuring the percent of the rainfall that is infiltrated and stored during the continuous simulation and measuring maximum outflow to the river during this period. Additional water quality assessment is disregarded in this study due to a lack of data.

(3) Functional resilience: The hydraulic performance of each scenario is assessed during the continuous simulation using several indicators such as maximum velocity in the pipes, the number of flooded manholes, accumulated flood volume, flood duration, etc. (Table 5.2).

(4) Public acceptance: I also considered public acceptance (stakeholders or decision-makers) by ranking the solutions based on their preference. In this study, I supposed that traditional centralized alternatives that rely mostly on the network of pipes have a higher acceptance.

Table 5.2: *Additional indicators, their category, symbols and units*

Indicator Category	symbol (Units)	Note
Technical and construction concerns (structural resilience)	Avg_D (m)	Average diameter of all pipes in the network
	max_D (m)	Maximum diameter of all pipes in the network
	Avg_E (m)	Average buried depth of all pipes in the network
	max_E (m)	Maximum buried depth of all pipes in the network
	$L_{D>1}$ (%)	Percent of the length of pipes with diameter>1 m to the total length of pipes
	$L_{E>2.5}$ (%)	Percent of the length of pipes with buried depth>2.5 m to the total length of pipes
Environmental sustainability	S_t (mm)	Percent of the rainfall that is infiltrated and stored during the continues simulation to the total precipitation
	$max_{flow,t}$ (l/s)	Maximum flow between all outlets during the continues simulation
Functional resilience	$max_{vel,t}$ (m/s)	Maximum velocity in the pipes during the continues simulation
	$avg_{vel,t}$ (m/s)	Mean value of velocity in the pipes during the continues simulation
	$N_{f,t}$ (-)	Number of flooding manholes during the continues simulation
	F_t (m^3)	Accumulated flood volume from sewer manholes during the continues simulation
	$Avg_{h\_f,t}$ (h)	Mean value of flood duration of all flooded manholes during the continues simulation
	$max_{h\_f,t}$ (h)	Maximum value of flood duration between all flooded manholes during the continues simulation
Stakeholders acceptance	Stk_{Rank} (-)	Rank of solutions considering public acceptance

5.5 Results and discussion

IIn the first scenario, for each layout, 100,000 simulation evaluations are defined as the termination criterion in the BORG optimization engine. Considering two seconds for each simulation, the whole optimization procedure for each layout took approximately 2.5 days (10 days for all layouts) using a personal laptop: Intel Core i7 with a 2.8 GHz dual-core CPU and 16 GB random access memory (RAM). For the second scenario, all layout and DC variables are optimized. In this case, 800,000 simulation evaluations are defined as the termination criterion, leading to a computational time of approximately 18 days. The Pareto optimal solutions identified by the proposed framework for all scenarios are shown in Figure 5.3. This figure accurately demonstrates the relation between LCC, DC and total sustainability for the presented test case.

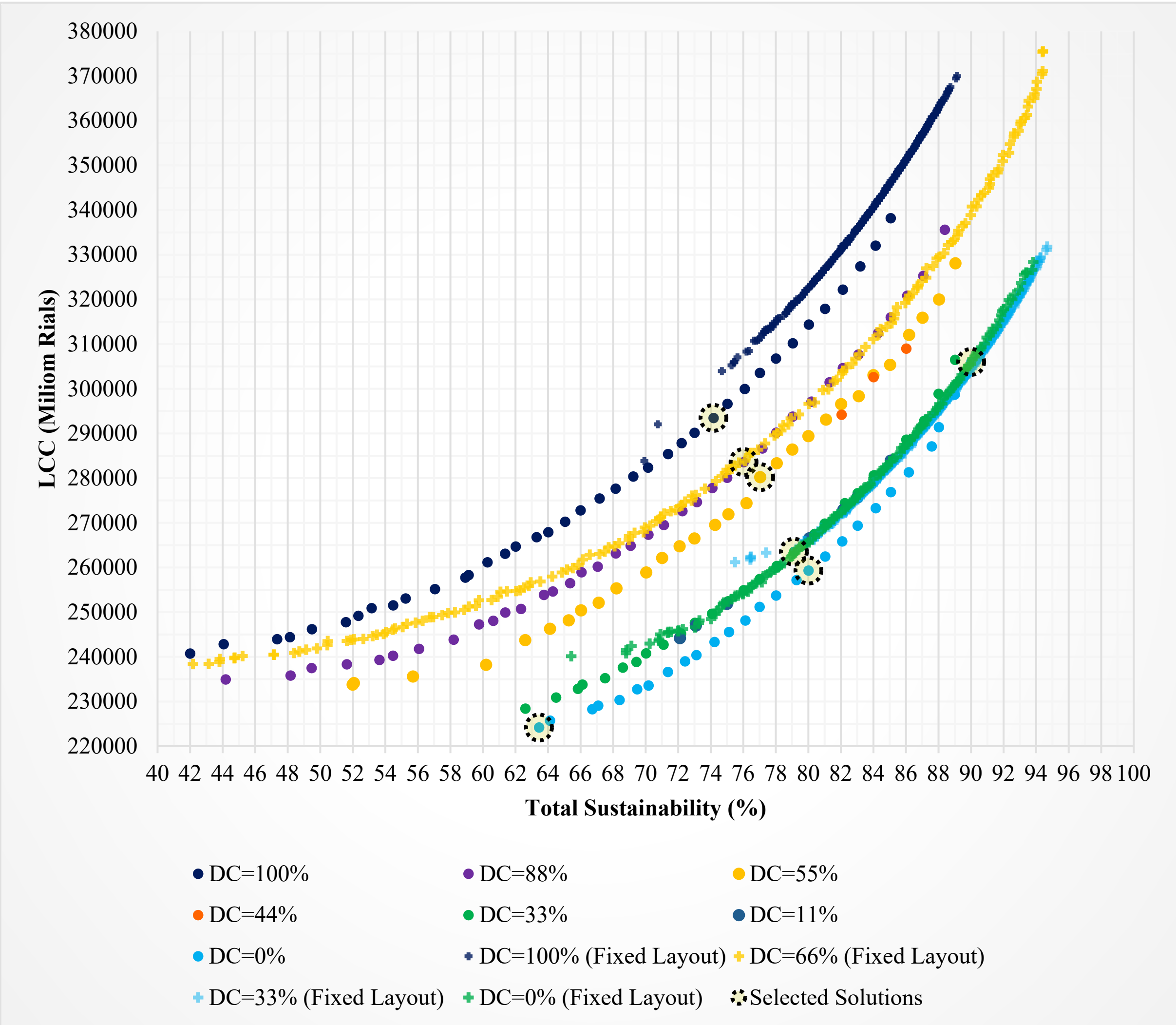


Figure 5.3: *Pareto optimal solutions identified by the proposed simulation-optimization framework*

5.5.1 Influence of (de-)centralization

As a first analysis, I look at the influence of DC on the Pareto front. For the fixed layout scenarios, DC has two main effects. First, all solutions with a certain DC are dominated by solutions with a lower DC. This means their lower DC values are beneficial. This is more obvious for the most centralized alternative (e.g., compare DC=100% and DC=55%). However, the more decentralized scenarios (DC=33% and DC=0%) are close to each other. This indicates that more structural decentralization does not cause a recognizable additional benefit. The same trend is observed for the second scenario, where the layouts are not fixed. The distance between Pareto fronts is high for more centralized solutions and decreases as DC decreases. Second, the range of solutions discovered by the MOO engine is highly dependent on the DC. As can be observed in Figure 5.3, the broadest range of solutions belongs to the layout with DC=66% (total sustainability ranging from 40 to 95), followed by DC=33%, DC=0%, and DC=100%. As a result, a very centralized or very decentralized layout might restrict the feasible search space. Therefore, more investigation is needed to determine whether there is a meaningful and insightful relation between the feasible search space or the number of solutions on the Pareto fronts and the system flexibility.

5.5.2 The role of flexibility

My next analysis examines the role of flexibility in the layout configuration on the quality of solutions for the optimum HGBGI. As seen in Figure 5.3, for a fixed DC, all the optimum solutions of the second scenario (layout optimized) dominate solutions of the first scenario (fixed layout). The solutions of the second scenario with DC=88% (two outlets) practically overlap with the solutions of the first scenarios with DC=66% (four outlets). Particularly, for DC=100%, solutions from the second scenario are observed to explore a broader range of objectives than the solutions from the first scenario. Only for DC=33% are the results of the two scenarios close together. This similarity might be explained by the well-designed layout of the first scenario for this DC. Overall, we conclude that joint global optimization, as is possible with our formulation, is advisable. Here, we omit the fixed layout cases (scenario one) from further analysis.

5.5.3 Final decision making

From the Pareto front of the second scenario, seven solutions have been selected to proceed to the final decision-making stage. Five solutions with different DC and LCC lower than 300,000 million Rials and a total sustainability greater than 70%, also the solution with the lowest LCC and the greatest total sustainability, are chosen from the Pareto fronts. Table 3 presents the indicator values, entropy weights, TOPSIS scores, and final solution rankings. The results show that the solution with DC=33% has the greatest score (rank one) among all the selected solutions. Figure 5.4 shows this design and its specifications. Other selected solutions (DC =100%, 88%, 55%, 0%) are represented in Figures. 5.5-8. The TOPSIS scores for solutions with ranks one to five are relatively close. This suggests that most of the solutions on the Pareto front can provide satisfactory quality. It must be noted that all of the above conclusions are based on a specific case study, and therefore, no general conclusion can be made. However, the proposed framework can be applied to different case studies with completely different situations without any restriction.

Table 5.3: *Results of indicators, entropy weights and the final ranking*

Indicators (units)	Solutions							
	DC= 100%	DC= 88%	DC= 55%	DC= 33%	DC=0% (a)	DC=0% (b)	DC=0% (c)	Entropy weight
LCC (M. Rials)	293490	283590	280230	263530	259320	**224200**	306030	0.0012
Rel (%)	98.99	99.05	98.49	98.94	99.1	88.91	**1**	0.0002
Res_{Fun} (%)	80.19	83.63	86.74	85.28	86.19	77.4	**99.76**	0.0008
Res_{Str} (%)	61.71	68.93	74.55	79.96	81.76	81.74	**82.06**	0.0013
Sus_{Env} (%)	62.4	59.05	56.32	58.7	59.18	32.17	**84.19**	0.0080
DC (%)	**100**	88	55	33	0	0	0	0.0506
Avg_D (m)	0.42	0.42	0.43	**0.41**	**0.41**	**0.41**	0.43	0.0001
max_D (m)	1.5	1.5	2	**1.2**	**1.2**	**1.2**	**1.2**	0.0050
Avg_E (m)	1.95	1.94	1.91	**1.86**	**1.86**	**1.86**	1.89	0.0000
max_E (m)	6.95	6.95	6.19	5.15	**4.8**	**4.8**	**4.8**	0.0036
$L_{D>1}$ (%)	5.79	5.62	6.74	5.07	**4.51**	4.62	6.69	0.0031
$L_{E>2.5}$ (%)	14.04	14.55	15.58	14.83	**14.82**	15.13	16.07	0.0002
S_t (%)	43.66	43.06	40.83	44.18	44.02	36.07	**57.49**	0.0024
$max_{flow,t}$ (l/s)	6996	5736	5851	2578	2121	3275	**1731**	0.0319
$max_{vel,t}$ (m/s)	4.04	3.39	3.08	2.65	2.72	3.69	**2.25**	0.0048
$avg_{vel,t}$ (m/s)	1.09	**1.02**	1.08	1.03	1.03	1.21	1.04	0.0004
$N_{f,t}$ (-)	136	11	3	2	3	141	0	0.2671
F_t (m^3)	1228	64	23	2	3	3420	0	**0.3436**
$Avg_{h\_f,t}$ (h)	0.75	0.49	0.7	0.23	0.2	0.94	0	0.0786
$max_{h\_f,t}$ (h)	1.67	0.77	0.73	0.33	0.3	4.31	0	0.1601
Stk_{Rank} (-)	6	2	**1**	3	4	7	5	0.0371
TOPSIS Score	0.496	0.900	0.912	**0.943**	0.931	0.024	0.939	
Rank	6	5	4	**1**	3	7	2	

Figure 5.4: *Final design (Optimal DC=33%, LCC=263530 Million Rials)*

Figure 5.5: *Selected solution 1 (LCC = 293490 M. Rials, DC = 100% + GBI)*

Figure 5.6: *Selected solution 2 (LCC = 283590 M. Rials, DC = 88% + GBI)*

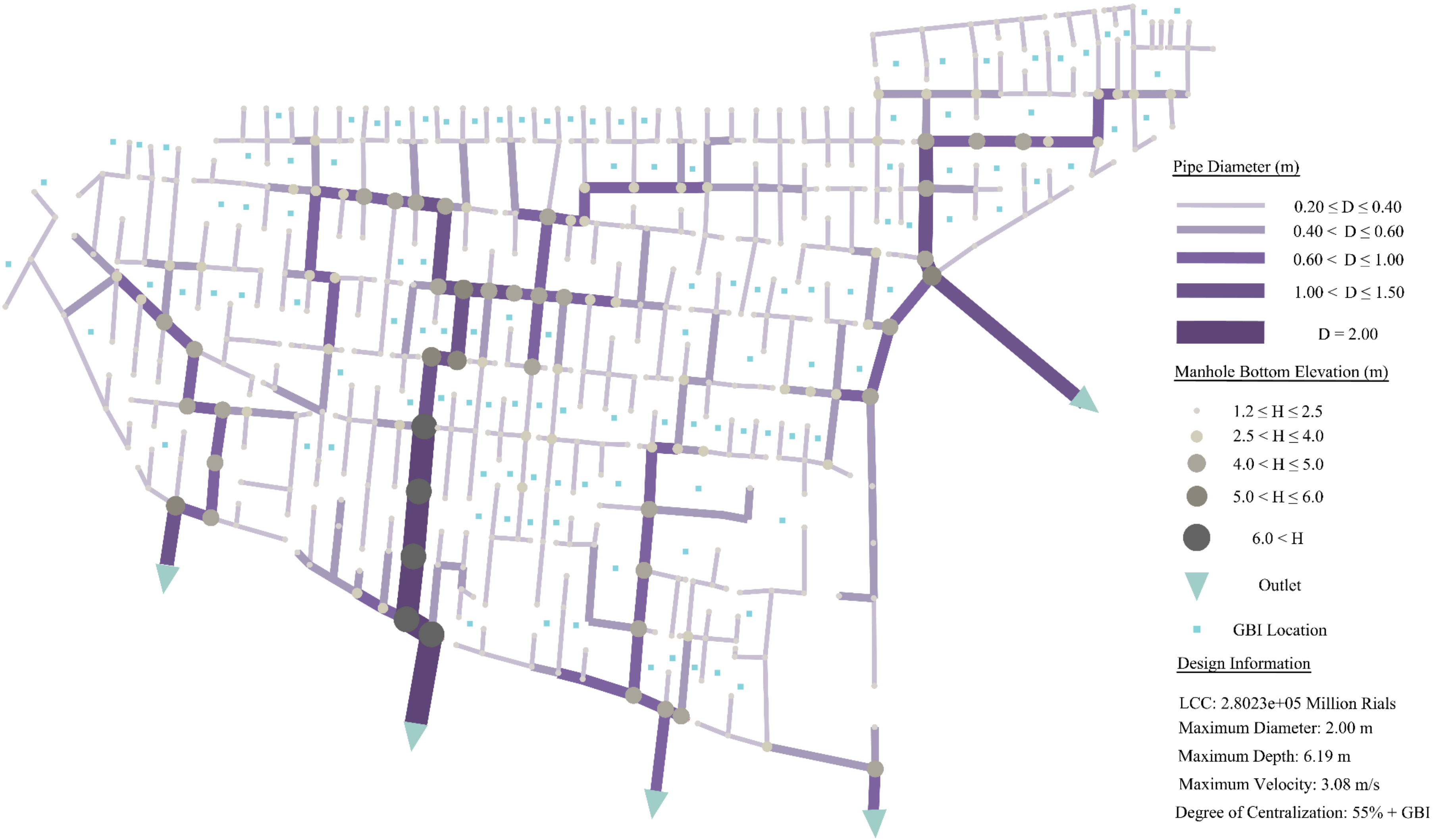


Figure 5.7: *Selected solution 3 (LCC = 280230 M. Rials, DC = 55% + GBI)*

Figure 5.8: *Selected solution 4 (LCC = 259320 M. Rials, DC = 0% + GBI)*

5.5.4 Stakeholders participation

As mentioned before, the proposed framework enables stakeholder involvement in the complex decision-making processes. In the problem definition stage, these stakeholders can suggest their desired technologies, such as different types of GBIs, treatment facilities or conventional systems. In the simulation-optimization stage, all possible combinations of these technologies are systematically generated and evaluated to provide a Pareto front of nondominated solutions. Finally, in the final decision-making stage, stakeholders participate in the process of selecting and ranking solutions. They can do so either by determining the weights of the indicators or by ranking the solutions based on their preference as an extra indicator. This approach might reduce the unfavorable procedural outcomes, resistance and conflicts that stakeholders often cause when they feel undervalued and unheard [128]. However, this framework restricts the domain of their contribution to only feasible, previously optimized and plausible solutions. We also calculate the rank of the solutions without considering the stakeholders' opinions (without including StkRank in Table 5.3 to calculate TOPSIS scores). The only change in comparison with the ranking in Table 5.3 is that the order of the first and second solutions is reversed.

5.5.5 Analyzing the structural resilience

Finally, the role of structural resilience in the layout configuration of the final design is investigated. Structural resilience is often neglected in the design of UDSs. However, it can play a significant role in system performance during extreme events. For instance, the fluvial flood that occurred in the study area in the winter of 2020 dramatically increased the groundwater level in the riversides. This issue resulted in the choking and blockage of several main pipes of the existing centralized wastewater collection network. As a consequence, several parts of the network, even far away from the riversides, were out of service. This led to wastewater spilling from manholes and overflowing in the streets, resulting in serious disturbances and health problems for citizens.

In Figure 5.4, the structural resilience of each outlet is presented. These values can be interpreted as the percentage of the total area not affected by any failure in that outlet. As an example, the minimum structural resilience among all the outlets is 79%, which belongs to outlet number 4. This means that only 21% percent of the total area is connected to outlet number 4. The critical pipes, which is the first pipe in each part of the system with a structural resilience less than 90%, are also demonstrated in Figure 5.4. The other pipes downstream of the critical pipes have less structural resilience. However, increasing the system's redundancy in weak points can resourcefully increase network resilience. This can be achieved, for example, by adding some loops or pipes that divert the flow direction to other parts of the network in emergency conditions. By doing so, the effects of single pipe failures can be significantly restricted. Four pipes out of 530 pipes in the case study are recognized as critical elements, as seen in Figure 5.4. Therefore, only by introducing four additional elements can a minimum (90% here) structural resilience be achieved for all pipes in the system. Interestingly, the structural resilience of outlet 9 is 84%; however, none of its upstream pipes has a structural resilience of less than 90%. The reason is that in this part, the stormwater is smartly collected from three different main collectors, as depicted in Figure 5.4.

5.6 Conclusion and outlook

This chapter introduced a multicriteria decision making platform for sustainable planning of urban drainage infrastructures considering different centralized or decentralized strategies. This platform encourages decision-maker engagement in all phases of the optimization procedure to increase the buy into the optimization results. The proposed framework is divided into three main stages. (1) System definition, in which the area under design is characterized, all desired types of urban drainage systems and technologies are introduced by decision makers, and the performance indicators are determined. (2) Simulation optimization, in which a multiobjective optimization problem is formulated based on simple reliability, resilience, and sustainability indices. Many hybrid design schemes (gray, green and blue elements with different sewer layouts and different degrees of (de)centralization) are generated and evaluated. This results in a Pareto front of nondominated solutions. (3) Final decision making, in which the selected solutions undergo a comprehensive assessment using a full range of indicators, including decision-maker preferences. Finally, a multicriteria decision-support technique is employed to rank the solutions.

The proposed framework was applied to design the stormwater management system of a section of Ahvaz, southwest Iran. Four simple indices to assess reliability, resilience (structural and functional) and sustainability were defined for the simulation-optimization problem. Additionally, fifteen mostly technical indices were proposed to rank the solutions. Two different optimization scenarios were considered: (1) the layouts were fixed, and (2) the layouts and their degree of (de)centralization were included in the optimization. The second scenario constructs an extremely complex nonlinear, mixed integer-real, highly constrained optimization problem with 1093 decisions, including layout configuration, pipe diameters, and green-blue infrastructure locations, in addition to three objectives: (1) life cycle costs, (2) total sustainability, and (3) degree of (de)centralization. During the simulation-optimization procedure, approximately 800,000 different HGBGI schemes were generated and evaluated.

The results confirm the ability of the proposed framework to handle many decisions, objectives, and indicators to solve the abovementioned complex optimization problem in a plausible time by delivering realistic solutions. The comparison between the results of the first and second scenarios demonstrates the significant role of the layout configuration and degree of (de)centralization on the optimum HGBGI. For a fixed level of sustainability, a significant reduction in life cycle costs might be obtained through a well-designed layout. The layout configuration can also determine the structural resilience of the system. The green-blue infrastructures that are used in this study can theoretically increase system flexibility. However, in future studies, tactics to increase the system redundancy in the design of layouts or sewers, such as introducing additional storage tanks, parallel pipes, or allowing for loops in critical zones, might be investigated.

The proposed framework facilitates stakeholders (decision makers) in decision making by involving them in different stages to decrease the conflicts between the stakeholders and optimization analysts. In the first step, alternative technologies are suggested, and in the last step, the solutions are prioritized or the weights of indicators are assigned. In future works, the proposed framework could be extended to resolve the conflict between multiple stakeholders with contradictory objectives [127, 144].

In the present study, simulations had to be limited to selected design storms. The choice of the design storms has an influence on the optimization result. We are planning to evaluate the robustness of the results against changes in rainfall scenarios in future studies considering different rainfall patterns and variations in the spatial distribution. We also plan to include multievent simulations (rainfall series) or use synthetic rainfall series considering climate change scenarios in the postprocessing step for the final decision making.

To apply the proposed framework to larger-scale problems, the computation time needs to be reduced. To do that, the application of meta-models instead of simulation software might be practical [145, 146]. Another alternative is to propose some indices based on the layout characteristics as a prescanning step in the simulation-optimization step to only consider layouts with a predefined acceptable condition for hydraulic design. Parallel processing can be used in all the above alternatives to accelerate the process. Although the proposed frameworks have been applied to design urban drainage systems, the general framework is relevant for planning other kinds of water resource systems as well.

Chapter 6. Epilogue

6.1 Summary of contributions

The main objective of this thesis was to assist sustainable UDSs planning. The general aim was to develop algorithms, methods and tools that aid in the mathematical interpretation of the trending concept of decentralization. The contributions made culminate in a generic Multi-Criteria Decision-Making platform to design modern urban drainage systems with the application to stormwater management systems.

The first contribution (Chapter 3):

At present, only a few approaches are available for generating and optimizing decentralized urban drainage alternatives, which are still far from real applications. To fill this gap, a layout generator, namely the *hanging gardens algorithm*, was developed to generate all possible (de)centralized urban drainage systems for both flat and steep terrains. To form a simulation-optimization framework, an optimization engine (A hybrid GA-Tabu optimization approach) is coupled with the proposed layout generator algorithm and with hydraulic simulation software (SWMM).

The resulting optimization tool forms a hard class non-linear mixed integer-real optimization problem with one objective function (construction costs) and many decisions, including the number and location of the outlets, layout configuration and the size of sewers. The model was then applied against a real case study, a section of the city of Ahvaz.

The second contribution (Chapter 4):

In the current literature, there is no tool or methodology for granting the interaction between the conventional network of pipes (GIs) and distributed measures (GBIs) in the design phase of UDSs. As a response to this deficiency, this chapter presented a simulation-optimization framework to optimize UDSs considering HGBGIs with different DC.

The proposed framework begins with generating different layouts with different DCs and hydraulically designs them using an adaptive algorithm. After introducing the feasible GBI to the model, a second optimization is performed to find the optimum distribution of GBIs in a way that minimizes the total life cycle costs of GBIs and pipe networks. This optimization is performed for each optimal gray-only system corresponding to the different degrees of centralization obtained in the previous step.

For the second optimization, as the layouts are fixed, the DC and layout decisions are removed from problem formulation. However, decisions related to type, location and size of GBIs are added to it. This setup significantly reduces the search space and, consequently, computation effort. Finally, the resiliency and sustainability of different scenarios are evaluated using several design storms that provide material for final assessment and decision-making.

The performance of the proposed framework is evaluated using the same case study, a part of the city of Ahvaz in Iran.

The third contribution (Chapter 5):

This contribution introduced an MCDM platform for optimal planning of hybrid urban drainage infrastructures. It can consider different centralized or decentralized strategies and encourages decision-maker engagement in all phases of the optimization to increase the buy-in to the optimization results.

Firstly, all desired types of urban drainage systems and technologies are introduced by decision-makers, and the performance indicators are determined. Next, a MOO problem is performed based on simple reliability, resilience and sustainability indices resulting in a Pareto front of non-dominated solutions. Finally, the selected solutions are undergone comprehensive assessment using a full range of indicators, including decision-makers' preference and an MCDM technique.

The proposed framework was applied again to design the stormwater management system of a section of Ahvaz. Two different optimization scenarios are considered: (1) the layouts were fixed, (2) the layout configurations were among the decisions. The second scenario constructs an extremely complex nonlinear, mixed integer-real, highly constrained optimization problem. It includes many decisions such as layouts configuration, pipes diameter and green-blue infrastructures location, besides three objectives; (1) life cycle costs, (2) total sustainability and (3) DC.

6.2 Summary of conclusions

The first contribution (Chapter 3):

This contribution showed that the proposed model exhibited good performance in exploring different degrees of centralization, generating realistic layouts and finding near-optimum solutions. A systematic optimization across all those degrees and their possible combinations of used candidate outlets down to the scale of a fully decentralized layout would, up to date, not be possible without the *hanging gardens algorithm*. Since the mathematical representation of generated networks is close to that of real sewer systems, the proposed framework introduces a comprehensive design package that can be employed for more realistic design as a superiority to existing conceptual models. The *hanging gardens algorithm* works on a random base and is self-adaptive so that any set of arbitrary decision variables, always lead to a feasible layout. Therefore, it can be coupled with any unconstrained metaheuristic as well as hydraulic simulation software.

The results suggested that structural decentralization can significantly reduce the construction costs, pipe sizes and invert depths in comparison with the centralized layout; however, after a particular DC (optimal DC), more decentralization might lead to a diseconomy of scale. The optimal DC totally depends on the case study specifications and problem setup. Besides, results demonstrated that structural decentralization could increase the functional resilience in the system.

The second contribution (Chapter 4):

This contribution demonstrated the performance of the proposed simulation-optimization framework to optimize UDS, considering the interaction of GIs and GBIs with different DCs.

The results showed that GBIs could significantly diminish the LCC of more centralized layouts. However, for the more decentralized layouts, the hybrid solutions were marginally more expensive than traditional solutions. Therefore, it can be understood that the GBIs have more impact on the more centralized network of pipes. The reason could be that capturing stormwater in each sub-catchment reduces the flow in all downstream parts of it while, in more decentralized networks, this only has effects on the part of the pipe network that is equipped with that GBI. The results also confirmed the poor functional resilience of hybrid green-blue-gray alternatives in comparison with traditional gray networks of pipes in facing severe rain-storms. The more a scenario had used GBIs, the more it was vulnerable.

On the other hand, hybrid green-blue-gray solutions showed better performance in environmental sustainability by a higher reduction in peak flow and higher storage and infiltrating capacity. This chapter also concluded that the optimal degree of centralization depends on the objectives, and it differs for cost, resilience and sustainability. Therefore, the optimization of new green-blue-gray UDSs should be done in a joint multi-objective framework for better decision making.

The third contribution (Chapter 5):

This chapter demonstrated the capacity of the proposed platform in handling many decisions, objectives and indicators for solving the above-mentioned complex optimization problem in a plausible time and delivering acceptable optimal scenarios. It has been manifested that many practical or technical concerns that usually cannot be regarded in general optimization frameworks but are crucial for decision-makers can be managed within the proposed framework to

(1) decrease the conflicts, (2) enrich the results of optimization with valuable experience of practitioners, and (3) increase the buy-in to the optimization results.

The results indicated that a pyramidal structure could explain the relationship between reliability, resilience and sustainability, which means resilience can be built upon reliability and sustainability can be built upon resilience. However, the LCC exponentially increases in all explored scenarios, with an increase in the total sustainability. The results demonstrate the significant role of the layout configuration and degree of centralization on the optimum HGBGI. For a fixed level of sustainability, a significant reduction of life cycle costs might be achieved through a well-designed layout. The layout configuration also can determine the structural resilience of the system.

6.3 Overall discussion

The implication for urban drainage planning in Iran

Thirst for development and rapid modernization in Iran has had provided a more advanced water management system than in most Middle Eastern countries. However, in the rush for infrastructure and technological development, less attention was paid to long-term environmental impacts, resulting in many serious adverse effects on the public health and environmental including drying lakes, rivers and wetlands, declining groundwater levels, land subsidence, water quality degradation, soil erosion, desertification and more frequent dust storms [147, 148].

Despite the adverse environmental and economic effects, the thirst for rapid technical and technological development (as opposed to sustainable development) is still the main driver of the country's development decisions [147]. Therefore, a paradigm shift in the current Iranian water management is indispensable to avoid the recurrence or expansion of similar issues in the future. At the outset, decision-makers in the country must appreciate the intricacy of the coupled human-natural systems to be able to develop water management solutions that have minimal secondary hostile impacts [147, 148].

The current thesis was an attempt to address the challenges mentioned above. Although the proposed frameworks have been applied to design urban drainage systems, the general ideas and concepts as presented in the following paragraphs, are relevant for planning other kinds of water resources systems.

Achieve optimality through decentralized solutions

The influence of the degree of centralization on the LCC and performance (reliability, resilience and sustainability) has been extensively investigated through this thesis. The *hanging gardens algorithm* developed in Chapter 3 can be modified and applied for optimal planning of any other network-based infrastructures even fully or partially looped networks such as water distribution networks.

Achieve optimality through hybrid solutions

The results of this thesis confirmed that hybrid solutions that combine advantageous elements and simultaneously complement deficiencies of conventional and new technologies might be the most promising approach for new infrastructure planning. To overcome the complexities made through this hybridization and find the optimal solutions among an infinite number of potential combinations of these technologies, the application of informatics tools and methods is inevitable. The general frameworks that are developed in Chapters 4 and 5 can be adjusted and employed for other urban water infrastructures planning such as sanitation systems and water distribution networks (by combining different water and wastewater treatment technologies and pipe networks with different degrees of (de)centralization).

Pursue reliability, resilience and sustainability during the planning phase

The importance of incorporating reliability, resilience and sustainability in infrastructure planning has been well acknowledged in the literature. The current water crisis in Iran is one example of disregarding these vital aspects, as has been addressed at the very beginning of this section. However, including many objectives to the problem, exponentially increases its complexity. Still, the proposed framework in Chapter 5 showed a satisfactory performance to handle problems with many goals and decision alternatives and to resolve the potential arising conflicts between scientists and stockholders. The proposed framework could readily be transformed to be applied in another era of water resources planning and management.

6.4 Outlook

In the following, five potential research themes to complement, expand and build upon the presented thesis are given.

Hybrid decentralized sanitation system planning and integrated water management systems planning

In the future, the presented frameworks in Chapters 3 to 5 could be extended to design and optimize separate hybrid decentralized sanitation systems. To do that, the *hanging gardens algorithm* needs to be extended to consider treatment facilities at outlets, pumping facilities. Different types of treatment facilities such as conventional wastewater treatment plants, on-site solutions, constructed wetlands in combination with different DC and layouts configuration, besides various often-conflicting objectives (e.g., costs, effluent quality to protect public and environment, public acceptance, system reliability and resilience) forms another notably intricate but exciting optimization problem.

Integrated urban water management is another area that might be pursued in the future based on the proposed frameworks within this thesis. Integrating the different components of urban drainage systems (e.g., storm and sewage collection conduits, wastewater treatment facilities and receiving waters) allows for more sophisticated management of stormwater and wastewater interventions [149]. As in an integrated framework, broader aspects of the system and the intercommunication between different components is focused, a variety of strategies and measures such as a wider range of GBIs, gray water recycling and wastewater reclamation schemes could be incorporated and assessed [149]. For example, in Chapter 4, if the integrated water system is taken into account, more expensive GBIs such as green roofs, bio-retentions and permeable pavements that provide many benefits like recharging groundwater, increasing urban amenity and alleviating the urban heat island effect, can be introduced as supplementary decisions to the optimization problem. As in an integrated urban water management framework, multiple stakeholders with contradictory objectives are involved, the proposed framework in Chapter 5 could be extended to resolve the potential conflicts.

Increasing the computation efficiency

The computation resources might restrict the applicably of the proposed frameworks, especially for integrated urban water infrastructure planning and management.

Optimization of integrated urban water systems could exponentially increase the computation expenditure as the number and type of decisions due to sizeable regional case studies and numerous possible measures for individual systems and the number of objectives due to multiple sectors' involvement are increased.

As a remedy, future studies could be equipped with model simulation parallelization techniques, surrogate modeling methods, further optimization tricks (e.g., using heuristic parameters as pre-screening of the generated layout before the hydraulic simulation) to reduce the number of time-consuming hydraulic simulations. Another possibility for this aim is reducing the level of model complexity or details.

Including uncertainties

All the presented frameworks in this thesis used a deterministic optimization approach that does not count for uncertainties. However, many uncertain parameters in UDS modeling could affect the reliability of model results significantly. Therefore, future research could investigate the effect of uncertainties associated with model parameters, the model aims due to changing regulations, loading conditions due to climate change and dynamic of cities (changing settlement patterns and shrinking or growing of cities, [149] on the optimal solutions.

Discovering the effect of different topography and settlement patterns on the optimal DC

The case study introduced in this thesis is characterized by flat topography in a highly urbanized region. Therefore, all results and conclusions made for optimal DC are case-specific and cannot be directly transferred to other areas with different specifications. Future studies could apply the proposed frameworks on other regions with topographic complexity ranging from flat to steep and different settlement distribution from sparsely to highly populated regions. The results of such studies would provide us with a clearer picture of optimal DC.

Integrating other aspects of resiliency

In Chapter 5, only two simple indices that represent only two types of threats are introduced to assess functional and structural resilience in UDS. However, to build resilience in the system, a comprehensive exploration of the system failure scenarios space is needed [134].

Developing other resilience indicators that can account for different types and combinations of threats can be pursued in future studies. Besides, enhancing system redundancy and flexibility to build resilience in UDSs can be addressed through many different design alternatives, such as introducing additional storage tanks, parallel pipes or allowing for some loops in critical zones. Future research is needed to investigate the trade-off between construction costs, building flexibility and redundancy in UDSs and different aspects of sustainability.

Considering existing infrastructures, retrofitting and rehabilitation strategies

The introduced frameworks in this thesis are applied to design a completely new UDS for the case study. However, all proposed frameworks in Chapters 3 to 5 could be modified to consider existing infrastructures and proposed optimal retrofitting and rehabilitation strategies.

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